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NATIONAL DAM SAFETY PROGRAM. INGHAMS DAM (INVENTORY NUMBER NY 1--ETC(U)
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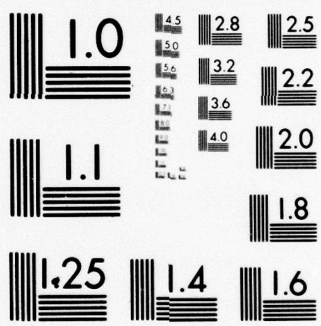

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Dam Safety	
National Dam Safety Program	Inghams Dam
Visual Inspection	Herkimer and Fulton Counties
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.

The examination of documents and visual inspection of Inghams Dam and appurtenant structures did not reveal conditions which would constitute a hazard to life or property. The dam has a number of problem areas which require further investigation and remedial action. These areas ~~are:~~ include:

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1) Additional hydrologic investigations are required to more accurately determine the site specific characteristics of the watershed. Using the Corps of Engineer's Screening Criteria for initial review of spillway adequacy, it has been determined that the spillway embankment would be overtopped for all storms exceeding approximately 24% of the Probable Maximum Flood (PMF) spillway capacity = 21,000 cfs. A flood wave analysis was not conducted since the location of the power plant near the dam was visually determined to be within any flood wave during dam failure. Also, several homes are located along the banks of the downstream channel. The spillway ~~is, therefore,~~ adjudged as "seriously inadequate", and the dam is assessed as unsafe, non-emergency. The classification of "unsafe" applied toa dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean that there appears to be a serious deficiency in spillway capacity, and if a severe storm were to occur, overtopping and failure of the dam could take place, significantly increasing the hazard to loss of life downstream of the dam. A detailed emergency operation plan and warning system should be developed and around-the-clock surveillance should be provided during periods of unusually heavy precipitation.

2) The stability analysis conducted in 1970 did not include the condition of ice loading pressures on the dam. Further, the results of the analysis conducted indicate that the location of the resultant falls outside the middle 1/3 of the base, with the development of tension at the heel for all cases investigated. Therefore, additional investigation is required to determine the type and extent of remedial action required.

3) The reservoir drain has not been operated since completion of the dam. The drain should be investigated and returned to operating condition, and

4) An inspection of the spillway should be conducted during no flow conditions to assess the integrity of the spillway.

Within 3 months, these investigations must be initiated and completion scheduled within 1 year of notification. Remedial action should then be completed in the following year.

The following deficiencies were observed which require remedial action:

1. Surface spalling observed on the downstream face and deterioration of the parapet walls should be monitored and repaired as required.

2. If erosion is initiated in the backfill at the north abutment of the spillway, then repair of the training wall at the base of the spillway should be completed.

3. Initiate a program of periodic inspection and maintenance of the dam and appurtenances. Document this information and develop an operations manual.

MOHAWK RIVER BASIN

INGHAMS DAM

HERKIMER AND FULTON COUNTIES, NEW YORK

INVENTORY NO. N.Y. 183

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

CONTRACT NO. DACW-51-79-C0001

NEW YORK DISTRICT CORPS OF ENGINEERS

MARCH, 1979

DISCLAIMER NOTICE

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
INGHAMS DAM I.D. No. NY 183
(formerly Kyser Lake Dam)
DEC #142D-572
MOHAWK RIVER BASIN
HERKIMER-FULTON COUNTY, NEW YORK

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PHASE 1 REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Inghams Dam (I.D. No. NY 183) (formerly Kyser Lake Dam)
State Located:	New York
County:	Herkimer - Fulton
Stream:	East Canada Creek (tributary of the Mohawk River)
Dates of Inspection:	October 16, 1978 and March 21, 1979

ASSESSMENT

The examination of documents and visual inspection of Inghams Dam and appurtenant structures did not reveal conditions which would constitute a hazard to life or property. The dam has a number of problem areas which require further investigation and remedial action. These areas are:

1. Additional hydrologic investigations are required to more accurately determine the site specific characteristics of the watershed. Using the Corps of Engineer's Screening Criteria for initial review of spillway adequacy, it has been determined that the embankment would be overtopped for all storms exceeding approximately 24% of the Probable Maximum Flood (PMF) spillway capacity = 21,000 cfs. A flood wave analysis was not conducted since the location of the power plant near the dam was visually determined to be within any flood wave during dam failure. Also, several homes are located along the banks of the downstream channel. The spillway is, therefore, adjudged as "seriously inadequate", and the dam is assessed as unsafe, non-emergency. The classification of "unsafe" applied toa dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean that there appears to be a serious deficiency in spillway capacity, and if a severe storm were to occur, overtopping and failure of the dam could take place, significantly increasing the hazard to loss of life downstream of the dam. A detailed emergency operation plan and warning system should be developed and around-the-clock surveillance should be provided during periods of unusually heavy precipitation.

2, The stability analysis conducted in 1970 did not include the condition of ice loading pressures on the dam. Further, the results of the analysis conducted indicate that the location of the resultant falls outside the middle 1/3 of the base, with the development of tension at the heel for all cases investigated. Therefore, additional investigation is required to determine the type and extent of remedial action required.

3. The reservoir drain has not been operated since completion of the dam. The drain should be investigated and returned to operating condition.

4. An inspection of the spillway should be conducted during no flow conditions to assess the integrity of the spillway.

Within 3 months, these investigations must be initiated and completion scheduled within 1 year of notification. Remedial action should then be completed in the following year.

The following deficiencies were observed which require remedial action:

1. Surface spalling observed on the downstream face and deterioration of the parapet walls should be monitored and repaired as required.

2. If erosion is initiated in the backfill at the north abutment of the spillway, then repair of the training wall at the base of the spillway should be completed.

3. Initiate a program of periodic inspection and maintenance of the dam and appurtenances. Document this information and develop an operations manual.

George Koch

George Koch
Chief, Dam Safety Section
New York State Department
of Environmental Conservation
NY License No. 45937

Clark H. Benn

Approved By:

Col. Clark H. Benn
New York District Engineer

Date: 8 August 79

Photograph #1

Overview of Inghams Dam
Downstream Face

Photograph #2

Overview of Spillway & Tailrace Channel
from top of dam

Photograph #3

Overview of Non-overflow Section
from top of dam

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
INGHAMS DAM I.D. No. NY 183
(formerly Kyser Lake Dam)
DEC #142D-572
MOHAWK RIVER BASIN
HERKIMER-FULTON COUNTY, NEW YORK

SECTION 1: PROJECT INFORMATION

1.1 GENERAL

a. Authority
The Phase 1 inspection reported herein was authorized by the Department of the Army, New York District, Corps of Engineers, to fulfill the requirements of the National Dam Inspection Act, Public Law 92-367.

b. Purpose of Inspection
Evaluation of the existing conditions of the subject dam to identify deficiencies and hazardous conditions, determine if they constitute hazards to life and property and recommend remedial measures where necessary.

1.2 DESCRIPTION OF PROJECT

a. Description of the Dam and Appurtenant Structures
Inghams Dam consists of a 480 feet long concrete gravity non-overflow section and a 205 feet long concrete ogee spillway. The maximum height of the dam is 125 feet. The upstream face of the dam is vertical. The crest of the non-overflow section is 12.5 feet wide and consists of two 2.5 feet high parapets. The downstream face of the dam is initially vertical, changing to a 3.2 to 10 batter and then changing to a 6.5 to 10 batter. The structure is founded on bedrock except for a limited section where a deep channel has been eroded in the rock (see drawing in Appendix G). The dam is founded on glacial till at this location.

The ungated concrete spillway is approximately 26 feet high and is also founded on bedrock. Flashboards 4.5 feet in height are used to provide additional storage capacity. The spillway crest is 8.5 feet below the top of the non-overflow section.

b. Location
Inghams Dam is located on the East Canada Creek between Herkimer and Fulton Counties and between the Towns of Manheim and Oppenheim, approximately 4.5 miles east of the Village of Little Falls.

c. Size Classification
The dam is 125 feet high and is classified as a "large" dam. (More than 100 feet).

d. Hazard Classification
The dam is classified as "high" hazard because of the presence of a number of homes along East Canada Creek.

e. Ownership
The dam is owned and operated by the Niagara Mohawk Power Corporation, 300 Erie Boulevard West, Building D-2, Syracuse New York, 13202, Tel: (315) 474-1511, Ext. 1869.

f. Purpose of the Dam
The dam provides storage for power development. In addition, the reservoir is used for recreational purposes.

g. Design and Construction History
The dam was constructed in 1911. The dam was designed by Viele, Blackwell and Buck, Consulting Engineers, 49 Wall Street, New York, NY. No engineering information pertaining to construction history was available.

h. Normal Operating Procedures
Water stored in the reservoir is used for the generation of electricity by the two turbines housed in the power plant approximately 600 feet below the dam. The water from the reservoir passes through the intake located in the gate house near the south abutment. Flow regulation is provided by an electrically operated slide gate at the gate house. Water passes through a screen to the intake chamber then to the 9 feet diameter steel penstock and through the surge tank to the power house. Flow is distributed to the turbines by hydraulically operated wicket gates. Flow not used in the generation of electricity is allowed to spill over the flashboards.

1.3 PERTINENT DATA

a.	Drainage Area (sq. mi)	278
	Height of dam (feet)	125
b.	Discharge at Dam Site (cfs)	
	Maximum known Flood	19,300
	Spillway at Design Pool (El. 665.5)	20,000
	Spillway at Maximum Pool (El. 665.8)	21,000
	Maximum Capacity of Reservoir drains	0
	Total Discharge, Max. Pool	21,000
	Average Daily Discharge	Unknown
c.	Elevation (ft. above MSL-Datum)	
	Top of Dam	665.8
	Design Pool	665.5
	Spillway Crest	657.3
	Tailrace Channel	539.0
	Invert Reservoir Drain Outlet	560.0
d.	Reservoir	
	Length of maximum Pool, miles	2.5
	Length of Shoreline (Spillway Crest) miles	5.7
	Surface area (Spillway Crest) acres	188

e.	Storage, (Acre-feet)		
	Spillway Crest		3,100
	Maximum Design Pool		4.500
	Top of Dam		4,600
f.	Dam		
	Type:	Gravity	Concrete
	Length (ft.)		480
	Upstream slope		Vertical
	Downstream slope		1:0.65
	Impervious Core	Concrete core wall keyed to the existing ground.	
	Crest elevation, ft.		665.8
	Crest Width, ft.		12
	Grout curtain		None
g.	Spillway		
	Type:		Ogee
	Length, ft.		205
	Crest Elevation MSL		657.3
	Upstream Channel		Not Visible
	Downstream Channel	Bedrock and in good condition.	
h.	Regulating Outlet	The 9 feet diameter penstock is regulated by an electrically operated slide gate at the upstream side of the dam. Maximum flow through penstock is 700 cfs. Most efficient flow is 600 cfs. An adjacent 6 feet diameter penstock has been plugged since the completion of construction.	
i.	Reservoir drain	2-6.5 feet diameter drains are inoperative.	

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Geology

The Inghams Dam is located in the northwestern portion of the "Hudson-Mohawk Lowlands" physiographic province of New York State. The province resulted from erosion along outcrop belts of weak rocks between the Aidrondack and Catskill Mountains. Generally, the province is of low elevation and relief. Bedrock in the vicinity of the dam is primarily Ordovician (500-435 million years ago) shales and sandstones which have been exposed by the southward and westward stripping off of Silurian and Devonian limestones. The present surficial soil deposits have resulted form glaciations during the Cenozoic Era (most recent 65 million year period), the last of which was the Wisconsin ice sheet approximately 11,000 years ago.

b. Subsurface Investigations

The "General Soil Map of New York State" prepared by Cornell University Agriculture Experiment Station indicates that the soils in the vicinity of the dam are Cazenovia and Mohawk. These soils are of glacial till origin and residuum from shale, siltstone limestone and small amounts of sandstone. They consist generally of stony or shaley silts and clays. Boulders are common. Rock outcrops in the spillway downstream channel and at both abutments of the dam. The internal drainage is poor and the rate of run-off dependent upon the degree of slope.

c. Dam and Appurtenant Structures

The dam was designed by Viele, Blackwell and Buck Consulting Engineers, 49 Wall St., New York, NY. Drawing Number R-3573, "Plan and Elevation of Dam and Spillway" is included in Appendix G. The design includes a concrete gravity dam and ogee spillway, founded on and keyed into bedrock till.

2.2 CONSTRUCTION RECORDS

The only information available concerning the construction of the dam is the year of construction (1911) and a newspaper report included in Appendix G. It was noted that a field change was made during construction from that shown on the drawing included in Appendix G. The south abutment of the dam was extended 63 feet southward from the intake structure and from that point, the concrete core wall was extended 108 feet southward. Elevations shown on this drawing are 6.7 feet higher than the actual U.S.G.S. elevations.

2.3 OPERATION RECORD

All information concerning discharges and maintenance is on file at the power house. No operating manual is available.

2.4 EVALUATION OF DATA

Some of the data presented in this report has been made available by Mr. Robert Levett of Niagara Mohawk Power Corporation. This information has been invaluable in the preparation of this report. All information gathered appears adequate and reliable for Phase 1 Inspection purposes.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General

Visual inspection of Inghams Dam and the surrounding watershed was conducted on October 16, 1978 and March 21, 1979. The weather was clear and temperatures ranged in the thirties. The reservoir level at the time of the first inspection was 662.3 (USGS) due to the presence of 4.5 feet high flashboards and 657.8 (USGS) during the second inspection due to the loss of the flashboards caused by spring runoff.

b. Concrete Non-Overflow Section

The non-overflow section has not been rehabilitated since its original construction. There are no signs of horizontal or vertical misalignment and no indication of distress or cracking. No seepage was evident at any location. Some surface spalling of the concrete was evident on the downstream face with the maximum depth estimated to be 3 inches and the averate depth of approximately 1 inch (see photographs #4,5 & 6). Surface spalling (see photographs #8,9 & 10) was also apparent on the parapet walls. No internal drainage system was provided. However, 6 borings were progressed in June 1970 to determine uplift pressures and these holes have remained open. No flow was observed from these holes. Boring logs for this work have been included in Appendix F "Stability Analysis". The intake structure for the penstock, located near the south end of the dam is in good condition. Some minor ice caused damage was apparent in the concrete walls of intake structure. (See photographs #8 & 12).

c. Spillway

The concrete ogee spillway section has not been repaired since its original construction. No signs of distress or movement were observed. Flow at the time of both inspections prohibited the complete examination of the spillway. A section of the training wall adjacent to the toe at the north end of the spillway was missing (see photographs #16 & 20). The remainder of this wall is in poor condition (see photographs #17, 18 and 21). The original purpose of the wall was to protect the north slope from erosion. Since the flow from the spillway generally does not reach the slope (the deteriorated wall provides little protection now) repair of the wall is not considered significant unless erosion is initiated in the vicinity of the north abutment of the spillway. Flashboards (4.5 feet high) observed during the first inspection were destroyed prior to the second inspection during a period of high runoff (see photographs #16 & 20). The tail race channel is exposed bedrock and appeared to be in good condition.

d. Regulating Outlets

Two penstock intakes are located in the non-overflow section of the dam. The 9 feet diameter penstock has one electrically operated slide gate located in the intake structure and is operational. (See photograph #8). An additional penstock intake was constructed for future electrical generating capacity. However, the expansion of facilities was not undertaken and this 6' diameter intake remains plugged.

e. Reservoir Drain

The two reservoir drains located at the center of the dam are 6.5 feet indiameter. The southerly pipe has been plugged and the northerly pipe has not been operated since the dam was completed. A manually operated butterfly valve is thought to control the flow of the northerly pipe and substantial seepage was observed discharging from this pipe. The controls for this valve have been destroyed. (See photograph #6).

f. Downstream Channel

The downstream channel is primarily bedrock formed and appears in good condition. (See photographs #19 & 22). A bridge located below the power house controls the flow of the channel. Some debris was observed in the downstream channel.

g. Reservoir

There are no visible signs of instability or sedimentation problems in the reservoir area.

3.2 EVALUATION OF OBSERVATIONS

No deficiencies were observed which would indicate that the dam is in imminent danger or which may develope into a hazardous condition. All deficiencies observed are of a minor nature and may be corrected by maintenance forces, with the exception of the inoperative reservoir drain.

SECTION 4: OPERATION AND MAINTENANCE PROCEDURE

4.1 PROCEDURE

Inghams Dam is a power dam for Niagara Mohawk Power Corporation. A 9 feet diameter steel pipe (penstock) carries water from the reservoir to the power plant approximately 600 feet below the dam. Flow through the penstock is controlled by an electrically operated slide gate located in the intake structure near the south abutment of the dam. Hydraulically operated wicket gates at the turbines control the flow from the penstock. Two 6.5 feet reservoir drains, one plugged and the other inoperative, have not been used since completion of the dam. The inoperative drain is believed to be controlled by a leaking butterfly valve, the mechanical controls of which have been destroyed.

4.2 MAINTENANCE OF DAM

There is no operation and maintenance manual for the dam. The dam is maintained in generally good condition. Some deterioration of the parapet walls and the downstream face of the non-overflow section was observed. A section of the deteriorated training wall at the northern toe of the spillway is missing.

4.3 MAINTENANCE OF OPERATING FACILITIES

The penstock intake slide gate is operational. The reservoir drain has not been operated since 1911.

4.4 WARNING SYSTEM IN EFFECT

There is no warning system in effect or in preparation.

4.5 EVALUATION

The structure is in need of some maintenance. A program of periodic inspection and maintenance of the dam and appurtenances should be initiated. This information should be documented for future reference. Also, develop an operations manual.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 DRAINAGE AREA CHARACTERISTICS

Inghams Dam is located on the East Canada Creek about 3.5 miles northeast of Manheim between Herkimer and Fulton Counties, New York. The drainage area at dam site is 278 square miles. The topography is characterized by mild to steep slopes interspersed by swamps and lakes.

5.2 ANALYSIS CRITERIA

Information on the PMF for Inghams Dam and its watershed was obtained from the UPPER HUDSON AND MOHAWK RIVER BASINS HYDROLOGIC FLOOD ROUTING MODELS prepared in 1976 for the New York District of the U.S. Army Corps of Engineers by Resource Analysis, Inc. In this study, the rainfall-runoff mathematical model HEC-1 was used to reconstitute the major historical floods and to simulate the Standard Project Flood (SPF). Probable Maximum Flood (PMF) was considered as twice the SPF.

The Inghams Dam and its watershed are located within the sub-areas 13 and 14 of the Mohawk River Basin, Little Falls, N.Y. to Mouth. The computed outflow resulting from one half PMF and PMF are 45,000 cfs and 89,000 cfs respectively (Appendix D). Since the storage is very little compared to inflow, the outflow is about the same as inflow.

5.3 SPILLWAY CAPACITY

The ungated concrete ogee spillway is 205 feet long and the maximum head possible between the crest of the spillway and the top of the dam is 8.5 feet. The lake level is raised by 4 feet 6 inches high collapsible flashboards erected on the crest of the spillway. The discharge capacity (source: discharge capacity curve supplied by Niagara Mohawk Power Corporation - see Appendix D) of the spillway is 21,000 cfs when the lake level is at the top of the dam (El. 665.8).

5.4 RESERVOIR CAPACITY

The capacities of the Inghams Lake at spillway level and at flashboard level are 3,100 acre-feet (AF) and 3,900 AF respectively. These volumes are above the intake level. The computed surcharge storage between the crest of the spillway and top of the dam is 1,500 AF which is equivalent to a runoff depth of .1 inch over the entire drainage area. Therefore, the effect of the reservoir in reducing the peak inflow is neglible for all practical purposes.

5.5 FLOODS OF RECORD

The highest flow occurred on March 18, 1936. The flow was 14,750 cfs and resulted in an elevation of 663.8 at the dam site. However, the highest flow at the dam site occurred as a result of an upstream dam failure on or about October 4, 1945. The flow was not recorded at the dam site but amounted to 19,300 cfs at the Dolgerville Gaging Station (drainage area = 261 square miles) approximately 3.5 miles upstream of Inghams Dam and floodwaters reached the elevation of El. 666.3 at the dam site. No records of low flows were recorded.

5.6 OVERTOPPING POTENTIAL

The PMF and ½ PMF peak outflows are 89,000 cfs and 45,000 cfs respectively, compared to spillway capacity of 21,000 cfs. Hence, the dam will be overtopped by 7.7 feet and 3.5 feet of water due to PMF and ½ PMF respectively.

5.7 EVALUATION

The spillway is considered inadequate to pass all storms in excess of 24% of the PMF. With regard to structural stability, the non-overflow section does not meet the minimum recommended factors of safety as defined by the Corps of Engineers. In the event of dam failure, the flood wave would pose a significant danger to the residents and workers within the power plant.

The spillway is, therefore, adjudged as seriously inadequate, and the dam is assessed as unsafe, non-emergency.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

The visual inspections did not indicate any signs of major distress in connection with the dam.

b. Design and Construction Data

No design computations or construction information prior to completion of the dam in 1911 are available. However, a structural stability analysis was conducted by Uhl, Hall & Rich, Consulting Engineers, Boston, Massachusetts in 1970. The results of this investigation are as follows:

Case 1 - Normal Water surface El. 663.3 Tailwater 563.3
Case 2 - Design Flood (20,000 cfs) El. 665.3 Tailwater 563.3
Case 3 - Probable Maximum Flood (100,000 cfs) El. 675.5 Tailwater 573.3
Case 4 - Seismic Condition - Normal Water El. 663.3 Tailwater 653.3
Earthquake Force = 0.05g horizontal

All cases assumed uplift as shown in Plate I Appendix F, which was the actual measured uplift conditions.

Case	Shear Friction Safety Factor	Location of Result from Toe	Overturning Safety Factor
1	5.75	19.6	1.35
2	5.60	17.8	1.30
3	4.88	7.0	1.10
4	5.04	14.5	1.24

These results indicate that the structure does not meet the minimum recommended factors of safety for overturning and location of resultant as defined by the Corps of Engineers "Guidelines" for all cases analyzed. The middle 1/3 of the base ranges from 26.3 to 52.7. Since all resultants analyzed fall outside the middle 1/3, tension will result at the heel. Also, no ice loading conditions were investigated. Further information concerning the stability analysis is included in Appendix F.

It is recommended that additional structural stability investigations be conducted and remedial measures instituted if necessary to achieve the minimum requirements for stability.

c. Operating Records

No operational problems were reported which would influence the stability of the structure.

d. Post-Construction Changes

To aid in the structural stability analysis, 6 borings were progressed in June 1970 on the downstream face of the non-overflow section to determine the actual uplift pressures at the base of the dam. These boring logs have been included in Appendix F. Analysis of the water levels in the borings indicated that the actual uplift pressures were lower than uplift pressures generally assumed in design (100% at heel to 100% tailwater). The reduced uplift considerable improved the predicted factors of safety.

e. Seismic Stability

The dam is located in Seismic Zone 2. A seismic analysis was conducted in Case 4 described above.

SECTION 7: ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety
The Phase 1 inspection of Inghams Dam revealed that the spillway is "seriously inadequate" and outflows from any storm in excess of 24% of the PMF would overtop the dam, resulting in a significant increase of the hazard to downstream residents. For this reason, the dam has been assessed as unsafe, non-emergency.

In addition, the structural stability analysis of the non-overflow section indicates that the factors of safety for all cases fall below the minimum requirements of the Corps of Engineers.

b. Adequacy of Information
The information available is adequate for Phase 1 inspection purposes. It should be noted that the design and construction information is extremely limited.

c. Urgency
The following investigations should be initiated within 3 months and completed within 1 year of notification: a detailed hydraulic/hydrologic analysis using the site specific characteristics of the watershed, structural stability including ice loading conditions, and investigation of the reservoir drain and its return to operational status. Remedial action as a result of these investigations should be completed in the following year. The remaining recommended measures described below should be completed during the next construction season.

d. Need for Additional Investigation
Investigate the hydraulic/hydrologic character of the watershed, and the structural stability of the dam including the case concerning ice loading pressures. Investigate the condition of the existing reservoir drain and institute remedial measures to restore the drain to its proper operational capacity. Observe the spillway under no flow conditions. The NYS Department of Environmental Conservation Dam Safety Section will be available to assist in this investigation. Telephone: (518) 457-6310.

7.2 RECOMMENDED MEASURES

a. Results of the required investigations will determine the remedial measures necessary.

b. Surface spalling observed on the downstream face and deterioration of the parapet walls should be repaired as required.

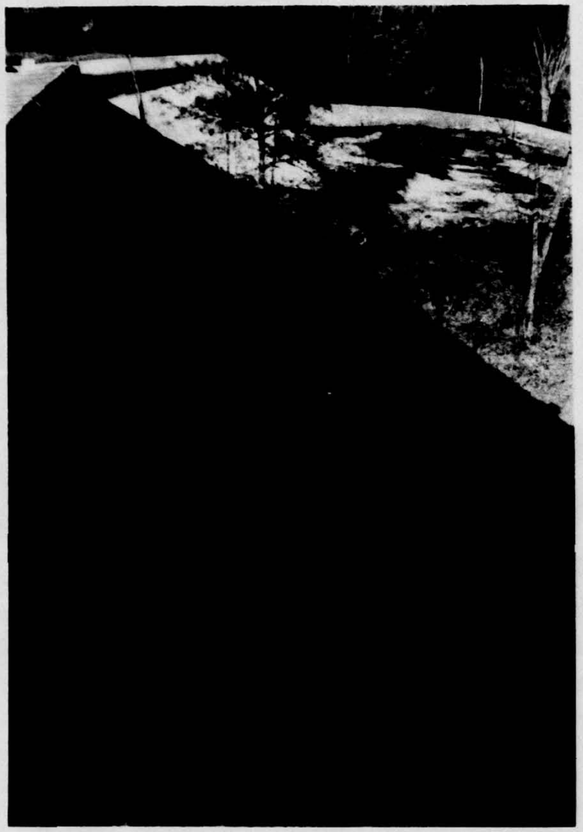
c. Repair the spillway training wall at the base of the spillway if erosion is initiated at the north abutment.

d. Initiate a program of periodic inspection and maintenance of the dam and appurtenances. Document this information for future reference. Also, develop an operations manual.

e. A detailed emergency operation plan and warning system should be developed. Also, around-the-clock surveillance should be provided during periods of unusually heavy precipitation.

APPENDIX A

PHOTOGRAPHS



Photograph #4

Non-Overflow Section Looking North

Photograph #5

Non-Overflow Section Looking South

Photograph #6

Non-Overflow Section viewed from top of dam
note leaking reservoir drain

Photograph #7

Old Photograph (undated)
probably during construction

Photograph #8

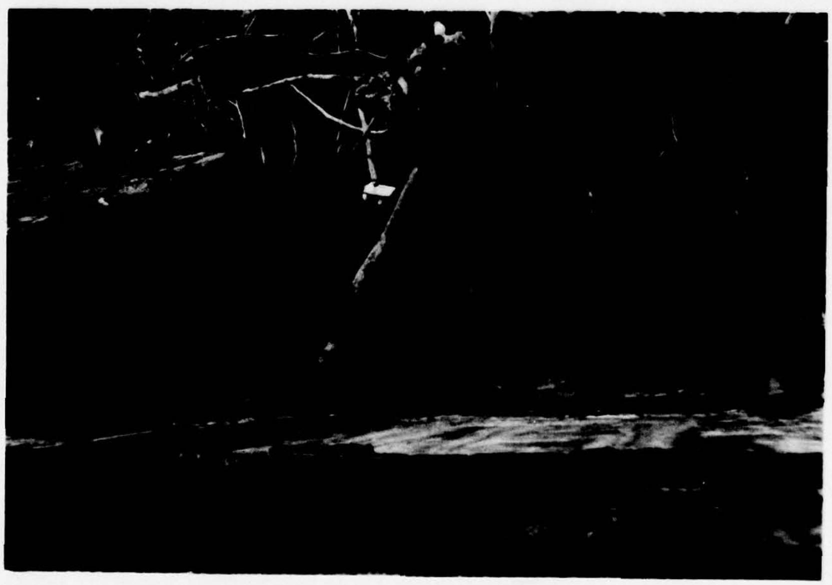
Top of Non-Overflow Section Looking South
at Intake Structure

Photograph #9

Top of Non-Overflow Section Looking South
note deterioration of parapet walls

Photograph #10

South Abutment of Non-Overflow Section Looking South



Photograph #11

South Abutment of Non-Overflow Section
downstream face as viewed from top

Photograph #12

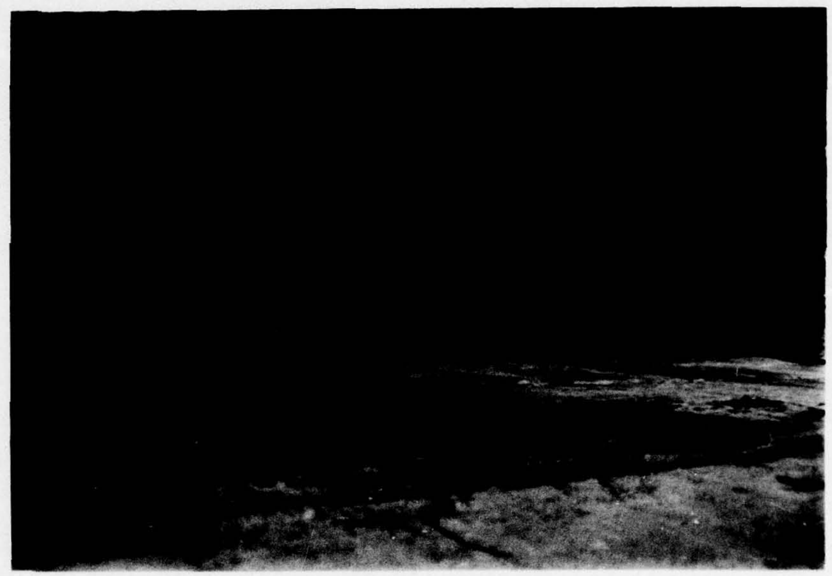
Upstream Face of Non-Overflow Section
and Intake Structure

Photograph #13

Trash Screen at Upstream Face
of Intake Structure

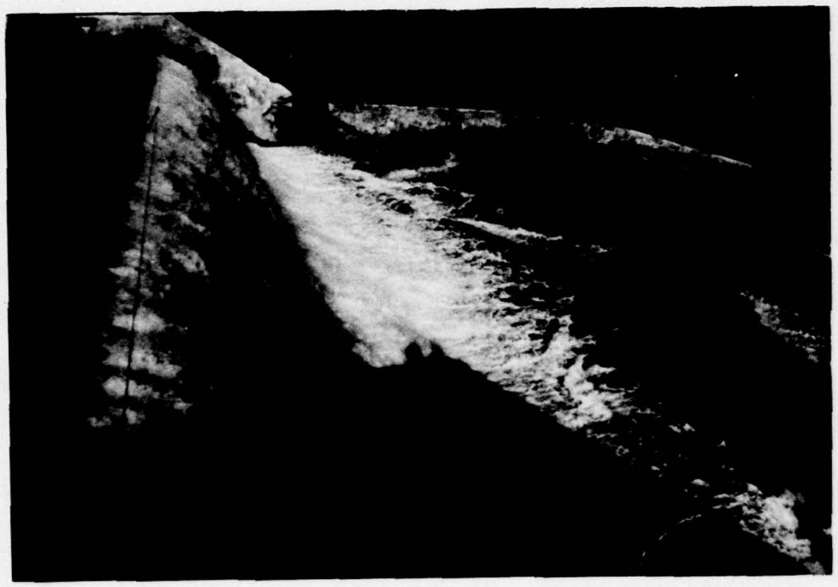
Photograph #14

South Spillway Wall, as Viewed from Top of Dam



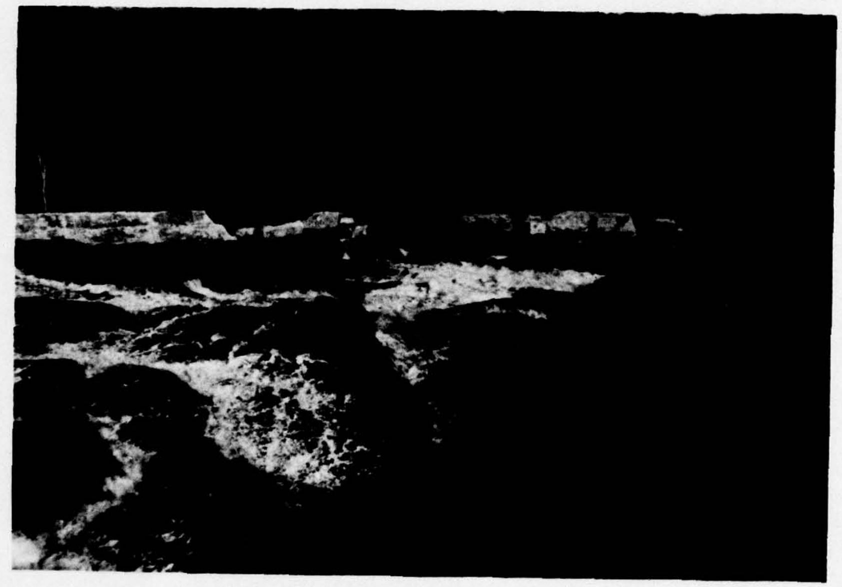
Photograph #15

Intersection of South Spillway Wall and Non-Overflow Section, as viewed from top of dam



Photograph #16

Ogee Spillway with flashboards looking north



Photograph #17

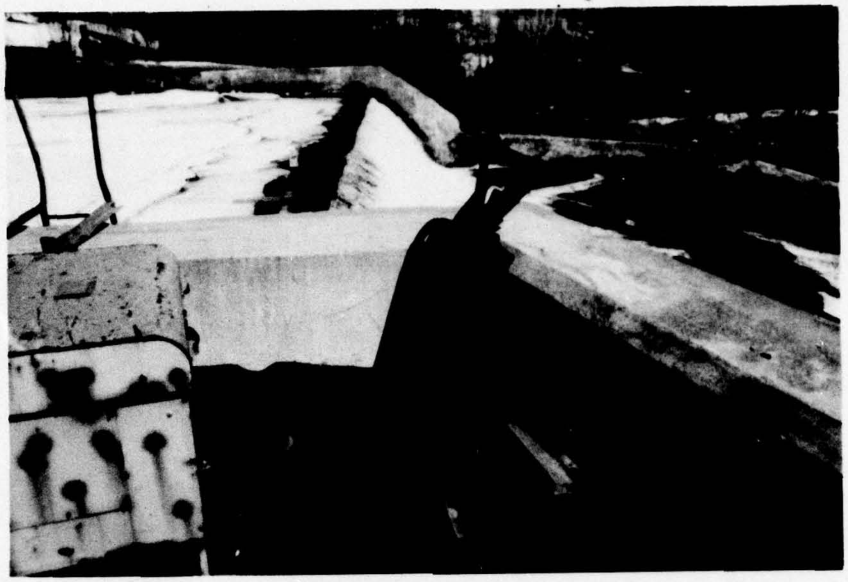
Tailrace Channel Below Spillway

Photograph #18

Area between Spillway Channel
and Non-Overflow Section Channel

Photograph #19

Downstream Channel below Non-Overflow
Section, Looking east



Photograph #20

Ogee Spillway without flashboards looking north
Note bent bars salvaged after failure of flashboards

Photograph #21

Channel Below Spillway where it enters
Non-Overflow Section Channel
(photo is adjacent to photo #1)

Photograph #22

Downstream Channel
as viewed from downstream bridge
below power house

APPENDIX B

ENGINEERING DATA CHECKLIST

Check List
Engineering Data
Design Construction Operation

Name of Dam Inghams Dam

I.D. # NY 183

Item	Remarks		
	Plans	Details	Typical Sections
Dam		-	YES
Spillway(s)	YES		
Outlet(s)	-	-	-
Design Reports	NONE		
Design Computations	NONE		
Discharge Rating Curves			
Dam Stability	1970 Report by Uhl, Hall & Rich		
Seepage Studies	None		
Subsurface and Materials Investigations	Borings from Dam stability Report		

Item	Remarks
Construction History	NONE
Surveys, Modifications, Post-Construction Engineering Studies and Reports	ONE REPORT TO STUDY UPLIFT PRESSURE
Accidents or Failure of Dam Description, Reports	NONE
Operation and Maintenance Records Operation Manual	All Records kept at Power House No manual

APPENDIX C

VISUAL INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

Name of Dam INGHAMS DAM (KYSER LAKE DAM)

I.D. # N.Y. 183

Location: Town MANHEIM & OPPENHEIM County HERKIMER & FULTON

Stream Name EAST CANADA CREEK

Tributary of MOHAWK RIVER

Longitude (W), Latitude (N) 74°-45'-59" / 43°-3'-42"

Hazard Category C

Date(s) of Inspection OCTOBER 16, 1978 March 21, 1979

Weather Conditions 30s SUNNY

b. Inspection Personnel ROBERT McCARTY, MUHAMMAD ISLAM ROBERT LEVETT, LOU PRATT

c. Persons Contacted ROBERT LEVETT, NIAGRA MOHAWK POWER CORPORATION, SYRACUSE, N.Y. 13202 TEL (315) 474-1511

d. History:

Date Constructed 1911

Owner NIAGRA MOHAWK

Designer VIELE BLACKWELL AND BUCK, 49 WALL ST. N.Y.C.

Constructed by unknown

2) Technical Data

Type of Dam CONCRETE AND MASONRY.

Drainage Area 278 SQUARE MILES

Height 125 feet Length 480 FEET

Upstream Slope VERTICAL Downstream Slope 1 : 0.65

a

2) Technical Data (Cont'd.)

External Drains: on Downstream Face NONE @ Downstream Toe NONE

Internal Components:

Impervious Core Concrete core wall at south abutment

Drains -

Cutoff Type -

Grout Curtain -

3) Embankment

None

a. Crest

(1) Vertical Alignment

(2) Horizontal Alignment

(3) Surface Cracks

(4) Miscellaneous

b. Slopes

(1) Undesirable Growth or Debris, Animal Burrows

(2) Sloughing, Subsidence or Depressions

(3) Slope Protection

(4) Surface Cracks or Movement at Toe

(5) Seepage

(6) Condition Around Outlet Structure

c. Abutments

No Earth Embankment

(1) Erosion at Embankment and Abutment Contact

(2) Seepage along Contact of Embankment and Abutment

(3) Seepage at toe or along downstream face

d. Downstream Area - below embankment

(1) Subsidence, Depressions, etc.

(2) Seepage, unusual growth

(3) Evidence of surface movement beyond embankment toe

(4) Miscellaneous

e. Drainage System

(1) Condition of relief wells, drains, etc. ______________________

____

____

____

____

(2) Discharge from Drainage System ______________________

____

____

4) **Instrumentation**

(1) Monumentation/Surveys NONE

(2) Observation Wells

6 borings processed in June 1970 were used to determine uplift beneath the dam and were left open

(3) Weirs NONE

(4) Piezometers NONE

NONE

(5) Other

5) **Reservoir**

a. Slopes O.K.

b. Sedimentation NONE Observed

6) Spillway(s) (including tail race channel)

a. General 205 feet long Ogee type approximately 26 feet high flashboards 4 feet in height are used to store extra water in reservoir

b. Principle Spillway CONDITION OF THE SPILLWAY COULD NOT BE DETERMINED BECAUSE WATER WAS FLOWING OVER FLASHBOARDS OVER SPILLWAY

c. Emergency or Auxiliary Spillway NONE

d. Condition of Tail race channel LOOKS CLEAN AS FAR AS VISIBLE.

e. Stability of Channel side/slopes Looks Stable.

7) Downstream Channel

a. Condition (debris, etc.) CLEAN

b. Slopes STABLE

c. Approximate number of homes THERE ARE 3 HOMES ABOUT 300 FEET BLOW THE DAM. ONE IS 20 FEET ABOVE CREEK BED AND OTHER 2 ABOUT 30 FEET ABOVE THE CREEK BED. THERE ARE A FEW MORE HOUSES FURTHER DOWNSTREAM.

8) Miscellaneous POOL IMMEDIATELY DOWNSTREAM OF DAM HAS SOME DEBRIS SUCH AS LOGS, DRUM ETCETRA.

9) Structural

a. Concrete Surfaces CONCRETE SURFACE is SPALLED AT DIFFERENT SECTIONS OF DAM. Particularly the downstream face max depth of spalling is 3 inches approx & average depth of 1 inch

b. Structural Cracking PARAPET WALL CRACKED IN PLACES.

c. Movement - Horizontal & Vertical Alignment (Settlement)

NONE OBSERVED

d. Junctions with Abutments or Embankments

good condition

e. Drains - Foundation, Joint, Face NONE

6 borings located near the toe of non-overflow section were progressed in June 1970 to determine uplift pressures these holes remain open

f. Water passages, conduits, sluices 9' diameter steel penstock to power house with elect. operated operational slide gate at intake and hydraulic operated wicket gates at the turbines

2 - 6'6" reservoir drains have not been operated since filling of the dam.

south pipe is believed plugged. North pipe has butterfly valve which is leaking

g. Seepage or Leakage

no seepage evident

h. Joints - Construction, etc.

generally good condition

parapet joints need repair

i. Foundation no problems observed

j. Abutments good condition

k. Control Gates operational for penstock to power plant, inoperative reservoir drain

l. Approach & Outlet Channels good condition

m. Energy Dissipators (plunge pool, etc.) rock channel

no problem

n. Intake Structures good condition

some limited ice damage to intake walls

o. Stability appears good

p. Miscellaneous A section of the training wall at the north end of the spillway adjacent to the toe of the spillway is missing no erosion was apparent in the area behind the wall (Rock foundation) repair if erosion is initiated.

APPENDIX D

HYDROLOGIC/HYDRAULIC

ENGINEERING DATA AND COMPUTATIONS

CHECK LIST FOR DAMS
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

AREA-CAPACITY DATA:

	Elevation (ft.)	Surface Area (acres)	Storage Capacity (acre-ft.)
1) Top of Dam	665.8 USGS		4600
2) Design High Water (Max. Design Pool)	665.5		4545
3) Auxiliary Spillway Crest			
4) Pool Level with Flashboards	661.8		3,860
5) Service Spillway Crest	657.3		3082

DISCHARGES

	Volume (cfs)
1) Average Daily	UNKNOWN
2) Spillway @ Maximum High Water	21,000
3) Spillway @ Design High Water	20,000
4) Spillway @ Auxiliary Spillway Crest Elevation	—
5) Low Level Outlet	NOT OPERABLE
6) Total (of all facilities) @ Maximum High Water	21,000
7) Maximum Known Flood due to natural flow	14,750
due to an upstream dam break	19,300

CREST: ELEVATION: 665.8

Type: CONCRETE

Width: base: 83 FEET; TOP; 12.5 FEET. Length: 480 FEET

Spillover CONCRETE OGEE 205 FEET LONG

Location NORTH SIDE OF DAM

SPILLWAY:

PRINCIPAL		EMERGENCY
657.3	Elevation	
CONCRETE OGEE	Type	
205 FEET	Width	
	Type of Control	
	Uncontrolled	
	Controlled:	
4.5 FEE HIGH FLASHBOARDS (COLLAPSIBLE)	Type (Flashboards; gate)	
-	Number	
-	Size/Length	
	Invert Material	
	Anticipated Length of operating service	
-	Chute Length	
26 FEET	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	

OUTLET STRUCTURES/EMERGENCY DRAWDOWN FACILITIES:

Type: Gate ______ Sluice ______ Conduit 2-6' Penstock ______

~~Shape~~ : ONE DRAIN PIPE WAS SHUT OFF AND THE OTHER IS

~~Size:~~ NON-OPERABLE. WATER LEAKS THRU ONE OF THE PIPES.

Elevations: Entrance Invert —

Exit Invert 560

Tailrace Channel: Elevation 539

HYDROMETEROLOGICAL GAGES:

Type : NONE

Location: ______

Records:

Date - ______

Max. Reading - ______

FLOOD WATER CONTROL SYSTEM:

Warning System: NONE

______

Method of Controlled Releases (mechanisms):

ONLY THRU PENSTOCK AND POWERHOUSE.

______

DRAINAGE AREA: 278 SQUARE MILES

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type:

Terrain - Relief:

Surface - Soil:

Runoff Potential (existing or planned extensive alterations to existing (surface or subsurface conditions)

NONE

Potential Sedimentation problem areas (natural or man-made; present or future)

NONE

Potential Backwater problem areas for levels at maximum storage capacity including surcharge storage:

NONE

Dikes - Floodwalls (overflow & non-overflow) - Low reaches along the Reservoir perimeter:

Location: NONE

Elevation: -

Reservoir:

Length @ Maximum Pool 2.5 (Miles)

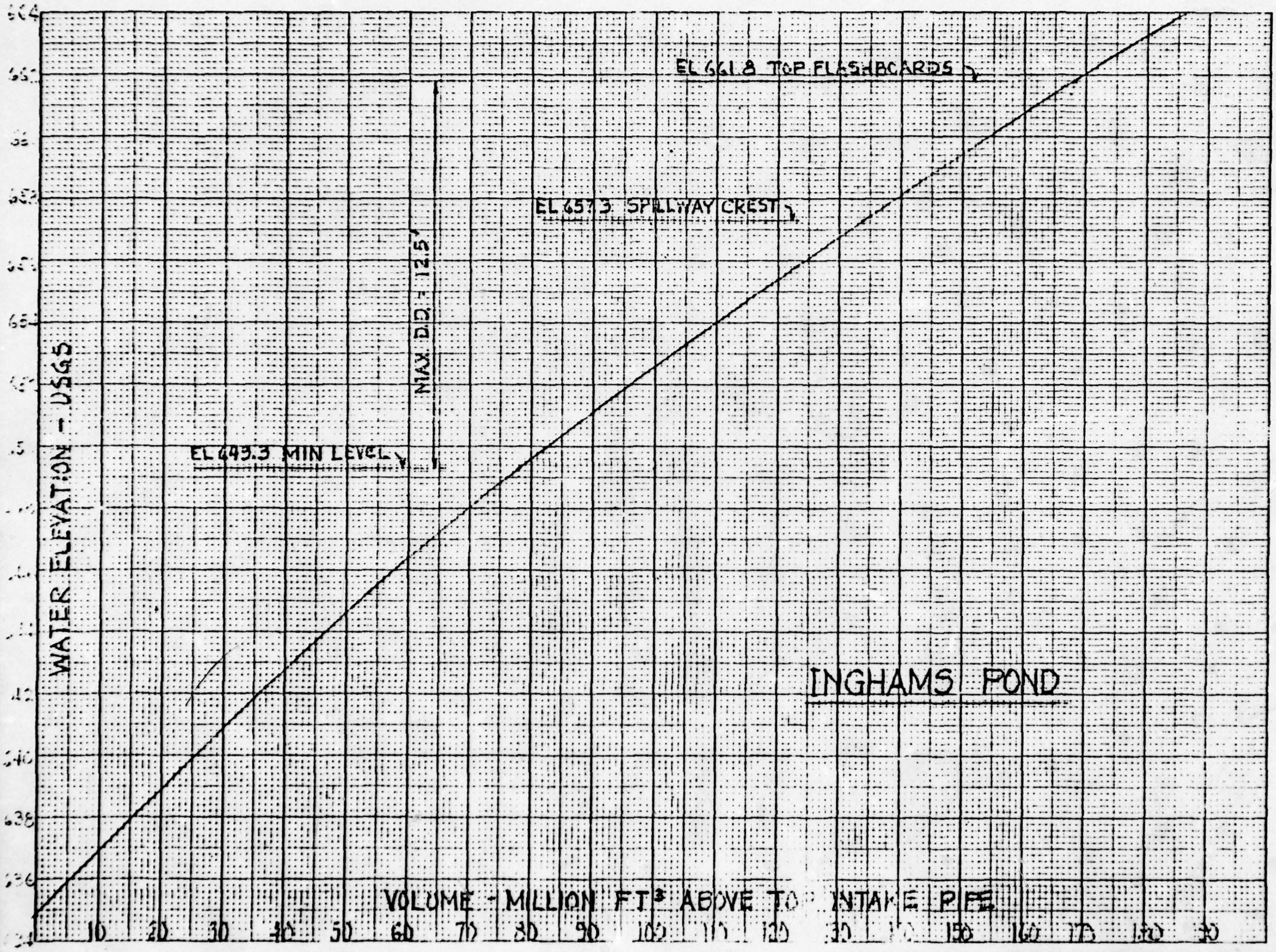
Length of Shoreline (@ Spillway Crest) 5.7 (Miles)

RESERVOIR DETENTION VOLUME

All figures from curve supplied by Niagara Mohawk Power Corporation.

All storage above intake level (634.8).

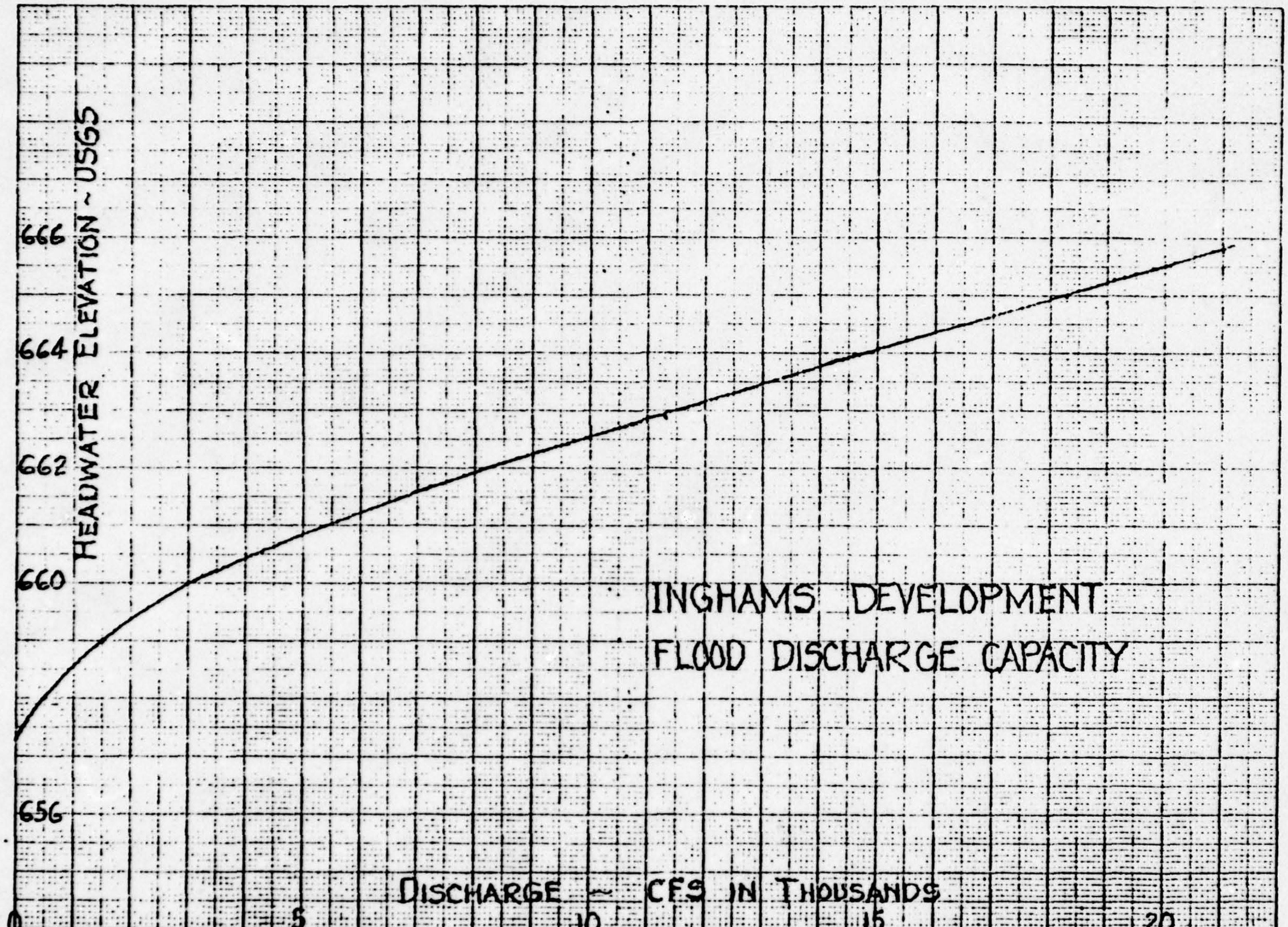
Elevation, ft. USGS	Volume in million ft^3	Volume in acre-feet	Remarks.
634.8	0	0	Top of intake pipe
636	5		
638	15.5		
640	25.5		
642	36.0		
644	46.5		
646	58.0		
648	70.0		
650	83.0		
652	96.0		
654	100.0		
656	124.5		
657.3	134.0	3082	Spillway crest
660	155.0		
661.8	168.0	3860	Top of flashboards
664	186.0	4278	
665.8	200.0	4600	Top of dam


INGHAMS POND
EL 661.8 TOP FLASHBOARDS
EL 657.3 SPILLWAY CREST
MAX. D.D. = 12.5'
EL 649.3 MIN LEVEL
WATER ELEVATION - USGS
VOLUME - MILLION FT³ ABOVE TOP INTAKE PIPE
10
20
30
40
50
60
70
80
90
100
638
636

SPILLWAY CAPACITY

All figures below are from Flood Discharge Capacity curve supplied by Niagara Mohawk Power Corporation.

Elevation, ft	Discharge, cfs
657.3	0 Spillway crest
659.0	1,500
660.0	3,000
661.0	5,500
661.8	7,600 ← Top of flashboards
662.0	8,200
663.0	11,500
664.0	14,700
665.0	18,300
665.8	21,000 Top of dam


INGHAMS DEVELOPMENT
FLOOD DISCHARGE CAPACITY
HEADWATER ELEVATION ~ USGS
666
664
662
660
656
DISCHARGE ~ CFS IN THOUSANDS
0
5
10
15
20

INGHAMS DAM

Drainage area = 278 square miles.

Name of stream : East Canada Creek

The following calculations are based on "Upper Hudson & Mohawk River Basins Hydrologic Flood Routing Models" study by Resource Analysis, Incorporated, for Corps of Engineers, New York District. Reference pages 97 -107.

Subdivision 16 of Mohawk River Basin (Little Falls N.Y. to Mouth) lies entirely in the drainage area of the Inghams dam. Major part of Subdivision 14 also lies in the same drainage area.

Area of subdivision 13 = 261 square miles

Area of subdivision 14 = 30 square miles.

But only 17 square miles of subdivision 14 lies in the drainage area of the Inghams Dam.

Standard Project Flood (SPF) = ½ Probable Maximum Flood (PMF)

½ PMF outflow at USGS gage # 3480 of East Canada Creek at East Creek is 46,206 cfs. and drainage area is 291 sq. miles.

$$\left(\frac{A_1}{A_2}\right)^{3/4} = \frac{\tfrac{1}{2}PMF_1}{\tfrac{1}{2}PMF_2}$$

or $$\left(\frac{278}{291}\right)^{3/4} = \frac{\tfrac{1}{2}PMF}{46,206}$$

or ½ PMF at dam site = 44,649 ≈ 44,700 cfs.

∴ PMF at dam site = 2 × 44,700 = 89,400 cfs.

OVERTOPPING

Length of dam = 480 feet $= L_1$

Length of spillway = 205 feet $= L_2$

$Q = CLH^{3/2}$, $C_2 = 4.13$, $L_1 =$ ft, $L_2 = 205$ ft

H_1 = height over dam

H_2 = Height over spillway.

PMF

Q = Total discharge

$H_2 = 8.5 \text{ ft} + H_1$

$$\underset{(\text{PMF})}{89{,}000} = C_1 L_1 H_1^{3/2} + C_2 L_2 H_2^{3/2}$$

$$= \cancel{3.0}3.33 \times 480 \times H_1^{3/2} + 4.13 \times 205 \times H_2^{3/2}$$

$$= 1598.4\, H_1^{3/2} + 846.65\, (8.5 + H_1)^{3/2}$$

$H_1 = 7.7$ feet

∴ the dam will be overtopped by 7.7 feet of water due to PMF.

$\frac{1}{2}$ PMF

$$45{,}000 = 1598.4\, H_1^{3/2} + 846.65\, (8.5 + H_1)^{3/2}$$

$H_1 = 3.5$ feet

∴ the dam will be overtopped by 3.5 feet of water due to $\frac{1}{2}$ PMF.

LIST OF REFERENCES

APPENDIX E

APPENDIX E

REFERENCES

1) U.S. Department of Commerce, Technical Paper No. 40, Rainfall Frequency Atlas of the United States, May 1961.

2) Soil Conservation Service, National Engineering Handbook, Section 4, Hydrology, August 1972 (U.S. Department of Agriculture).

3) H.W. King and E.F. Brater, Handbook of Hydraulics, 5th edition, McGraw-Hill, 1963.

4) T.W. Lambe and R.V. Whitman, Soil Mechanics, John Wiley and Sons, 1965.

5) W.D. Thornbury, Principles of Geomorphology, John Wiley and Sons, 1969.

6) University of the State of New York, Geology of New York, Education Leaflet 20, Reprinted 1973.

7) Cornell University Agriculture Experiment Station (compiled by M.G. Cline and R.L. Marshall), General Soil Map of New York State and Soils of New York Landscapes, Information Bulletin 119, 1977.

8) Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models, New York District Corps of Engineers.

APPENDIX F

STABILITY ANALYSES

UHL, HALL & RICH

Engineers

441 STUART STREET, BOSTON, MASSACHUSETTS 02116 • AREA CODE 617-262-3220
Southern Office: 1301 East Morehead Street, Charlotte, North Carolina 28204 • Area Code 704-376-4336

May 30, 1969 2225-2

Subject: Inghams Dam
Uplift Pressure System

Mr. R. J. DeStefano, System Structural Engineer
Niagara Mohawk Power Corporation
126 State Street
Albany, New York 12201

Dear Mr. DeStefano:

In compliance with your request we have reviewed the procedure previously outlined for drilling an exploratory hole in the tailrace area, to be followed by three pairs of observation holes through the concrete dam structure.

In answer to Mr. Clancy's question to you regarding the need for additional information pertaining to our proposed method of observation hole monitoring for determination of uplift pressures at the hole locations, we feel that the explanation and description of the planned procedure as follows would be adequate and sufficient for the F.P.C. to consider:

"It is planned to make an exploratory drill hole in the foundation immediately downstream of the dam at a point in the tailrace adjacent to the tallest section of dam. Cores will be taken at this hole to a depth of 25 feet below the base of the structure. Should this hole indicate normally sound foundation conditions, as is anticipated, then a series of observation holes will be made through the dam.

It is proposed that three sets of holes be cored through the dam from the downstream face. These three lines of holes will be located at the tallest section of the dam within the original river bed foundation. Spacing between the lines of holes will be approximately 50 feet, which will allow full coverage of the foundation at this deepest section of the structure. Each line of holes will consist of an angle hole and a vertical hole extending into the foundation a maximum of 3 feet. With the collar of these holes located in the vicinity of El. 573, which is above maximum expected tailwater, the vertical hole will contact the foundation at a point approximately 25 feet from the toe. The angle hole will then meet the foundation at about 60 feet from the toe or about 25 feet from the heel of the structure. The enclosed sketch will illustrate the location of the proposed observation holes.

From these holes, a piezometric height of water can be determined and translated into foundation pressures at the particular foundation contact points. It is planned to insert threaded pipe ends at the collars of the holes in order to allow attachment of pressure gages to be used should foundation drainage flows reach this elevation. These holes will normally be capped to prevent foreign material from entering the holes.

Readings would be taken at monthly intervals during the first year of operation of the monitoring system, assuming normal headwater - tailwater relation. Should operating conditions require a rapid or unusual drawdown of the reservoir, more frequent readings would be taken in order to establish trends in uplift conditions versus headwater elevations for the first year.

It is proposed that the readings at the angle holes and vertical holes respectively, would be averaged to be applied over the length of the base, applicable only at the deepest section of the structure. It is intended to treat the pressure at the upstream face of the structure as full headwater pressure, and likewise the pressure at the downstream face will be taken at tailwater pressure. Utilizing the average pressure readings determined from the observation holes, an uplift intensity can then be determined and compared to the extreme assumption of full headwater varying to full tailwater over the full base area. This calculated uplift intensity would then be used in determining the required safety factors of the structure against overturning under normal and flood conditions.

Should the results of these readings indicate that uplift intensities are still too high to provide the required minimum safety factors, then other remedial work would have to be considered to accomplish this."

In answer to Mr. Clancy's request that we provide some back up to you on the proposed procedure, we offer the following:

The principle involved here has been employed on many recent structures in a slightly modified way and has been found to be acceptable. The main variation in the method being that the same approach in general is taken except that provision is made to establish these check points at the foundation contact at the time of construction of the dam structures. A so called "pressure cell" is normally built in at selected points at the foundation contact and piping is then carried to some convenient point, usually to a grouting gallery or inspection gallery. At these locations drainage flows, if any, are measured and observed. If deemed necessary, pressure readings are taken. This system has been used by us on several dams and powerhouse structures.

The use of the actual system of drilling holes after construction has been used on many structures, but the majority of these have foundations which have been grouted during or after construction and also have a system of foundation drains installed downstream of the grout curtain. This eliminates the possibility of any qualitative comparison or projection of results at the Inghams site.

However, one installation of note where a system of drilled holes was installed through a dam and foundation which was not grouted or drained, this was at Holtwood Dam on the Susquehanna River. The dam is an overflow ogee spillway about 70 feet high. The foundation was a granitic schist containing some fault seams. Holes were drilled in several lines with 4 holes per line starting at seven feet back from the upstream face and spaced at fifteen feet between holes. All holes were vertical. Results of readings showed a deviation from the typical headwater to tailwater uplift assumption.

We trust that the above will suffice for your present needs. If further information is required please contact us.

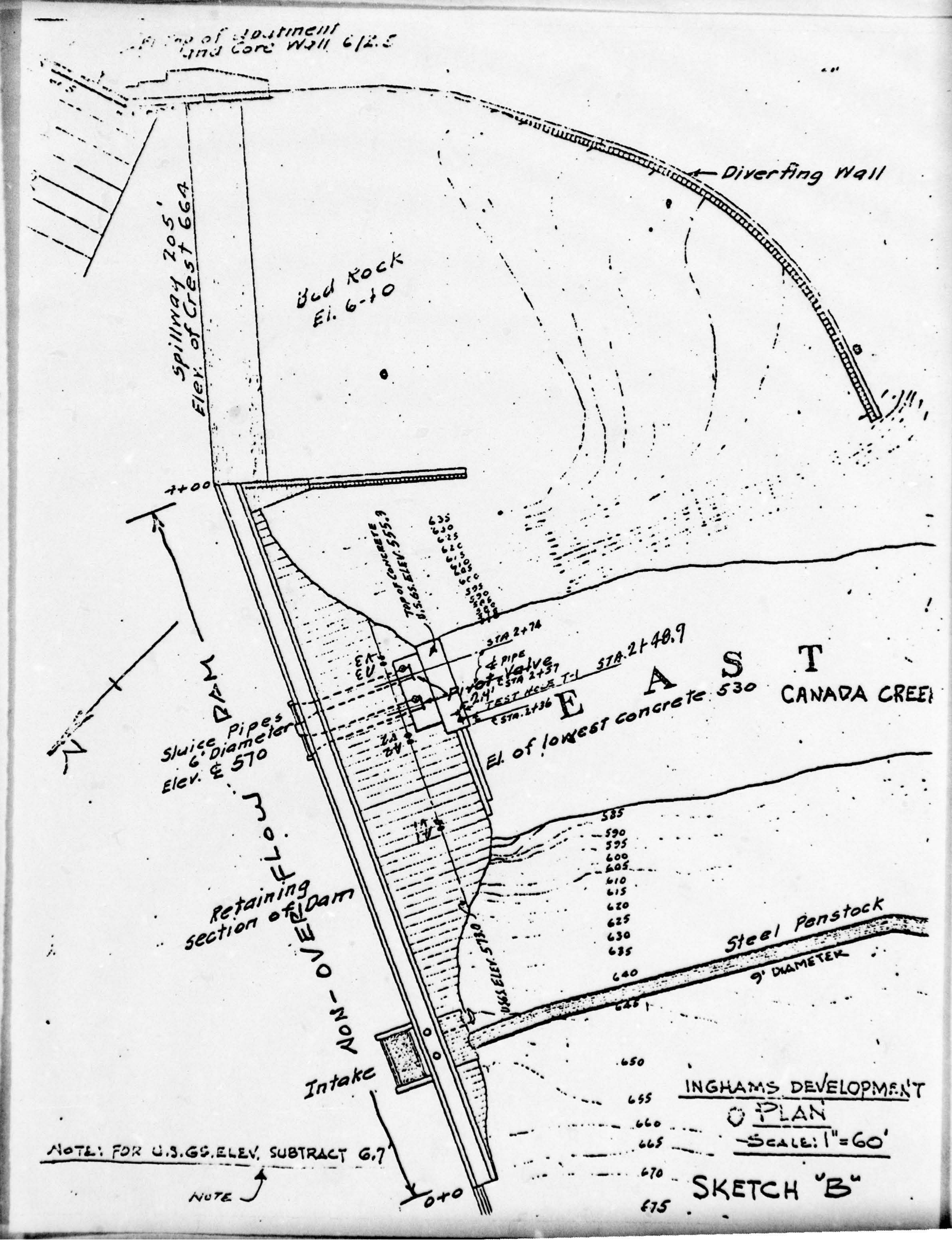
Very truly yours,

UHL, HALL & RICH

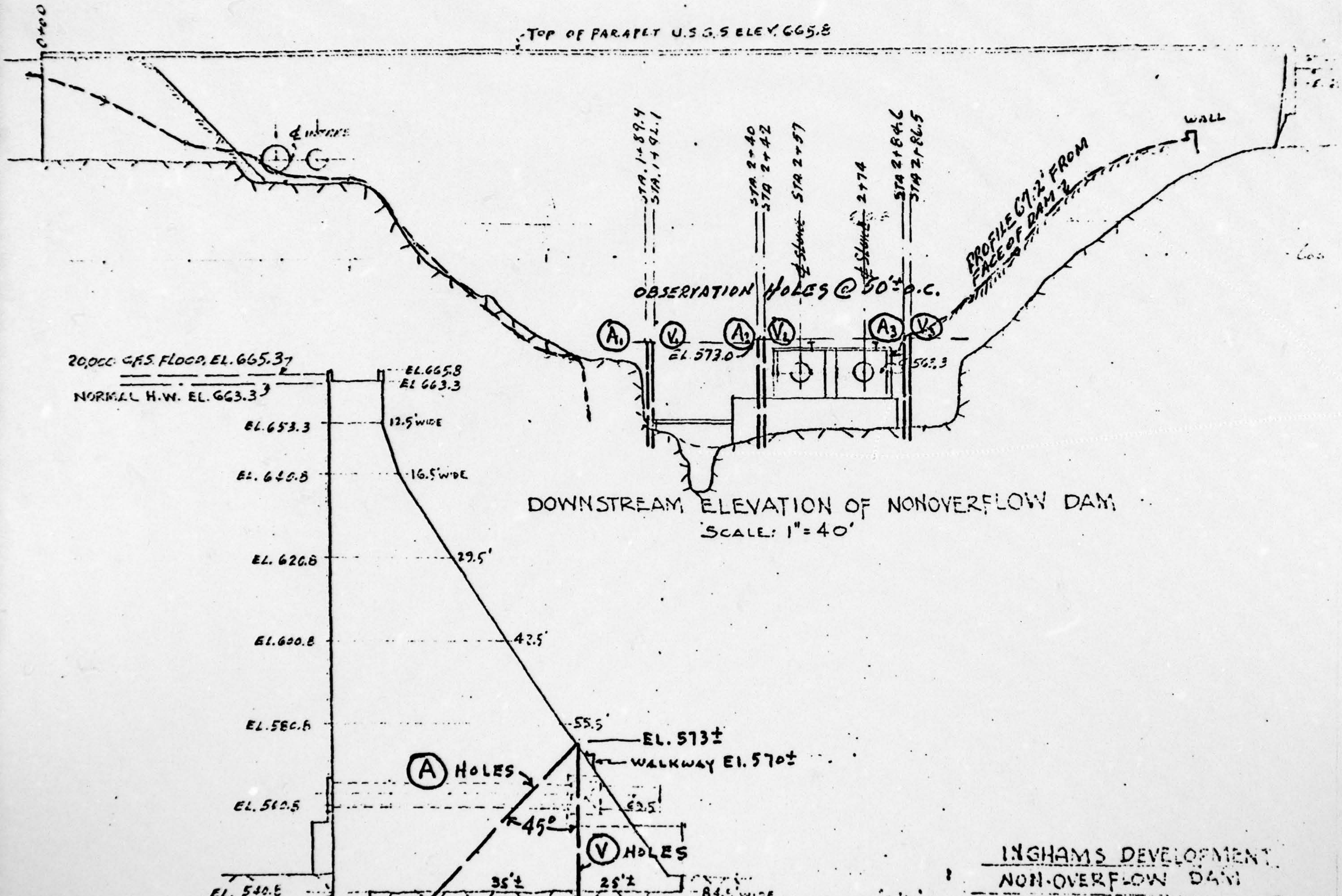
J. C. Matte

J. C. Matte

JCM/mf



El. Top of Abutment and Core Wall 612.5
Diverting Wall
Spillway 205'
Elev. of Crest 664
Bed Rock
El. 640
4+00
Top of Concrete U.S.G.S. Elev. 555.9
STA 2+74
STA. 2+40.9
Pipe Valve
Pivot Valve
STA 2+37
Test Hole T-1
STA 2+36
E A S T
CANADA CREEK
El. of lowest concrete 530
Sluice Pipes
6' Diameter
Elev. ℄ 570
NON-OVERFLOW DAM
Retaining section of Dam
Steel Penstock
9' DIAMETER
U.S.G.S. ELEV. 573.0
Intake
0+0
NOTE: FOR U.S.GS. ELEV. SUBTRACT 6.7'
NOTE
INGHAMS DEVELOPMENT
PLAN
SCALE: 1"=60'
SKETCH "B"

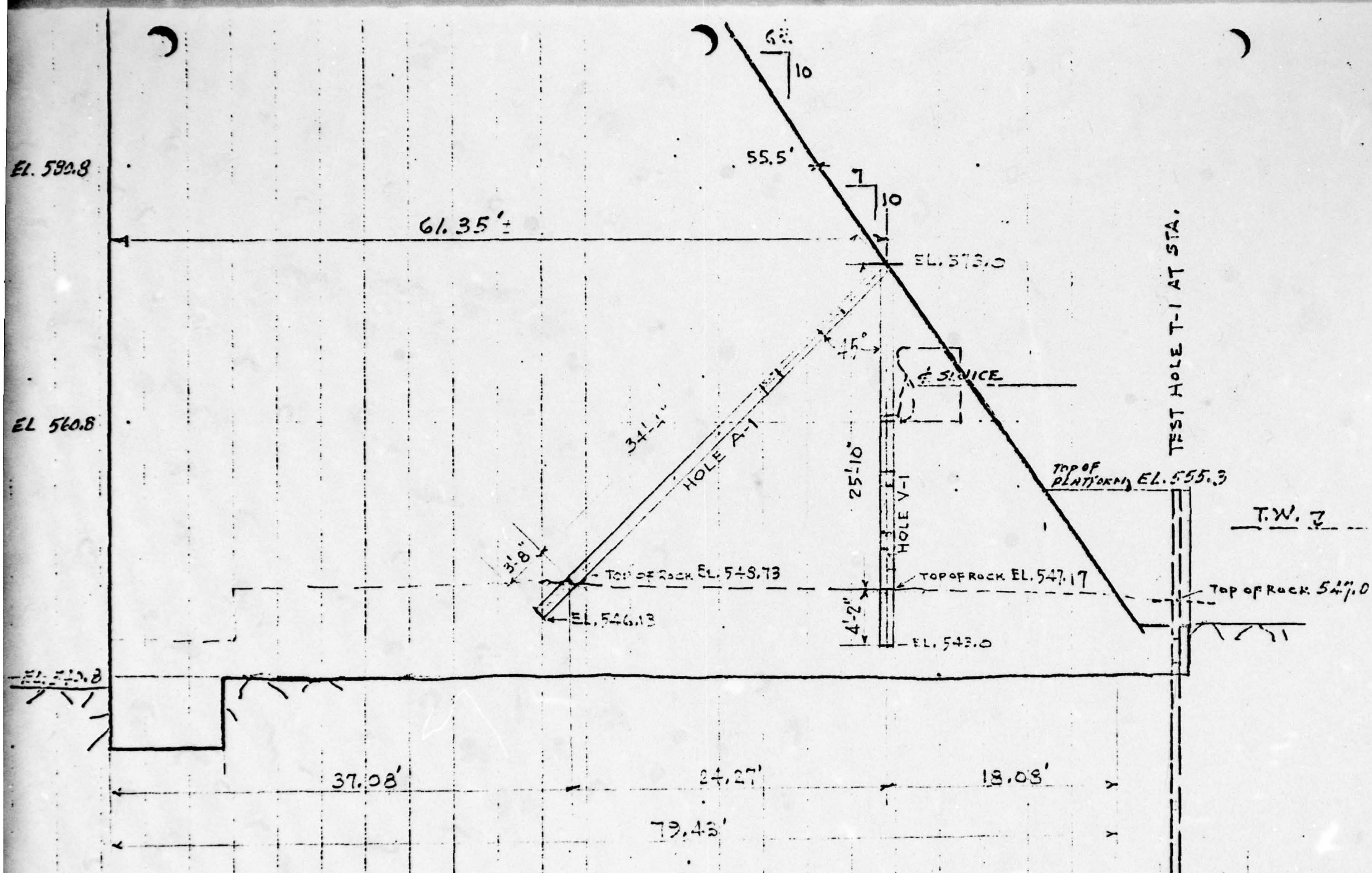


DOWNSTREAM ELEVATION OF NONOVERFLOW DAM
SCALE: 1" = 40'

SECTION THRU DAM
SCALE: 1" = 30'

INGHAMS DEVELOPMENT
NON-OVERFLOW DAM

PROPOSED TEST HOLES - SKETCH "A"



BOTTOM SECTION OF NONOVERFLOW DAM

CORE BORINGS VI & AI & T-1

SCALE: 1" = 10'

R.J.D.

NIAGARA MOHAWK POWER CORPORATION

INGHAMS DEVELOPMENT

EAST CANADA CREEK PROJECT NO. 2648

REPORT OF

STABILITY ANALYSIS

NON-OVERFLOW SECTION

INGHAMS DAM

UHL, HALL & RICH, ENGINEERS

Boston, Massachusetts

November 1970

2225-3

UHL, HALL & RICH

Engineers

441 STUART STREET. BOSTON. MASSACHUSETTS 02116 • AREA CODE 617-262-3220
Southern Office: 1301 East Morehead Street. Charlotte, North Carolina 28204 • Area Code 704-376-4336

November 16, 1970 2225-3

SUBJECT: Report of Stability Analysis
Inghams Dam Non-Overflow Section
East Canada Creek Project No. 2648

Mr. T. J. Brosnan, Vice President and Chief Engineer
Niagara Mohawk Power Corporation
300 Erie Boulevard West
Syracuse, New York 13202

Dear Mr. Brosnan:

We are pleased to transmit herewith our report on the stability of the non-overflow section of Inghams Dam.

The report includes a summary of the correspondence and discussions with the Federal Power Commission relating to the question of stability at this structure since the time of application for license in 1967.

As part of the stability investigation, an exploratory program was carried out through the concrete structure in order to determine actual uplift pressures at the base of the structure. It was determined by means of these exploratory or observation holes that actual uplift conditions were less than those required to be assumed in the absence of specific data.

It is intended that this report shall evidence the fact that the lessening of uplift forces employed in the stability analysis permits the satisfying of the minimum factors of safety established for the structure. In so doing, it is anticipated that no reinforcement of the structure will be required, as was considered earlier.

We trust that you will find this report to be complete and satisfactory.

Very truly yours,

UHL, HALL & RICH

J C Matte

J. C. Matte
Project Engineer

JCM/ss

UHL, HALL & RICH

Engineers

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STABILITY ANALYSIS REPORT

OF THE

INGHAMS DAM NON OVERFLOW SECTION

EAST CANADA CREEK PROJECT NO. 2648

SUMMARY

The East Canada Creek Project encompasses the Beardslee and Inghams Hydro developments, located on East Canada Creek in the towns of Manheim and Oppenheim, New York.

In July of 1967, the Federal Power Commission staff in reviewing the application for license of these developments, requested that a report on the stability of the structures at both projects be submitted. This report was required to include a flood study for the purpose of determining reservoir surcharge at each project.

In complying with the F.P.C. request, Uhl, Hall & Rich was engaged by the applicant to perform the stability analyses on the earth dam and concrete spillway structures at the Beardslee Development, and on the concrete spillway and non overflow dam sections at the Inghams Development. The required flood studies were performed by the applicant, Niagara Mohawk Power Corporation.

The report was prepared and submitted on September 27, 1967.

Further correspondence was received from the Chief of the Bureau of Power of the Federal Power Commission in a letter dated October 30, 1967.

Essentially, this letter contained the following opinions and comments:

1. The F.P.C. staff was of the opinion that in the event of dam failure there would be probable loss of life.
2. Spillway design flood should be based upon probable maximum precipitation rather than the 100 year flood derived by flood frequency analysis, as proposed by the applicant.
3. Once the spillway design flood was determined, stability analyses should be prepared for the normal reservoir loading condition and for the maximum flood reservoir loading condition.
4. Since no drains were provided at the base of the concrete structures, uplift pressures should be assumed as acting over 100 percent of the base areas rather than over two thirds of the base area as previously assumed. Any uplift assumptions different from those noted above should be verified by tests on the structures.
5. Information was requested as to whether soils properties as used in the stability analysis of the earth embankment at Beardslee Dam were based upon assumptions or soil tests.

Further studies were made by the applicant considering the statements and opinions as stated in the above letter. Both flood analyses and stability analyses were performed using the proposed criteria of the F.P.C.

The applicant took the position that flood analysis based on maximum probable precipitation assumptions for these existing developments was not appropriate. However, stability analyses were performed with uplift considered over 100 percent of the base area of the structures, with reservoir elevations as determined by a spillway design flood of 20,000 cfs. This latter figure was the magnitude proposed by the applicant as appropriate to

the developments under consideration.

A meeting was arranged and held at F.P.C. offices in Washington, D.C., with members of the F.P.C. staff, the applicant, and Uhl, Hall & Rich on March 7, 1969, to discuss in total all considerations relating to flood analysis and stability of the project structures.

The essential results of this meeting were as follows:

1. All previous questions regarding the safety and stability of the Beardslee Development structures were satisfied.
2. Factors of safety of the Inghams structures were discussed and it was determined that the spillway structure could be considered adequate in this regard while the non-overflow section of the dam had safety factors which were not satisfactory when full uplift was applied.
3. It was determined that methods of improving the factor of safety against overturning of the non-overflow section should be investigated. It was agreed that a satisfactory range of safety factor would be 1.25 or greater for normal conditions, including a design flood of 20,000 cfs. For a check of extreme conditions a factor of safety of 1.0 would be satisfactory for reservoir conditions corresponding to a maximum probable precipitation flood of 100,000 cfs.

Studies and preliminary cost estimates were made of various methods of dam reinforcement required for improvement of stability factors. Included among these were drilled in prestressed rods and rockfill embankment to be placed at the downstream face of the structure. The latter appeared to offer the only practical solution.

In recognition of the fact that uplift forces were of significant influence in the determination of safety factors, it was decided to investigate actual uplift conditions at the base of the structure and compare these to conditions assumed.

A proposed plan was developed to drill a total of six holes through the structure commencing at the downstream face at an elevation above normal expected tailwater. Three of these holes were to be vertical and three were to be angle holes inclined downward in the upstream direction. These holes, paired off as one vertical and one angle hole at about fifty feet between pairs would provide a piezometric measurement of uplift pressure at the points where the drilled holes intersected bedrock at the base of the dam. It was intended that individual pressure readings at each hole could be averaged and applied as actual uplift forces at the base of the structures. If these actual pressures were less than the intensities assumed on the conventional full headwater to full tailwater gradient as recommended by the F.P.C., then they would be applied in the stability analysis. In any case uplift pressures obtained would be applied over 100 percent of the base area.

The applicant submitted the proposed plan to the F.P.C. by letter dated July,11, 1969 requesting that they review and comment on the proposed procedure.

A reply to the above request was received from the F.P.C. by letter dated August 5, 1969. The commission staff had determined that the proposal to determine actual uplift pressures was satisfactory to them, and stated also that the stability analyses should include the normal water surface condition, seismic condition, the probable maximum precipitation flood condition, and and other loading condition which may be critical.

Specifications for the drilling work were prepared and a contractor selected for the work. It was considered preferable not to attempt this work under severe winter conditions and actual performance of the work was necessarily deferred until spring of 1970. The work was commenced in May and completed during the latter part of June 1970.

Daily water level readings were taken at each hole at completion and continually on a daily basis thereafter through July 2, 1970. Since, by that time it could be observed that either a steady state condition or a diminishing water level condition was commencing in the holes, it was then decided to take readings on a weekly basis.

The stability analysis results as included in this report utilized the actual uplift readings from these test holes. Since actual results proved to be less critical than the assumed uplift loading it was possible to obtain stability factors required by the F.P.C. as minimums, thereby alleviating the need to provide additional means of reinforcing the non-overflow dam section against the forces assumed in the anlaysis.

OBSERVATION HOLES

The six observation holes through the dam were drilled with rotary drills of standard NX size. Cores of the concrete structure and bedrock at the base of the structure were obtained and examined. All cores indicated sound concrete at all locations. Penetration into bedrock was limited to a maximum of about five feet at all holes. Excellent recovery of core was made in both concrete and bedrock at all holes.

The collar of all six holes was set at El. 573.0 nominally. All water surface elevations observed within the holes were referenced to this elevation.

With the exception of hole V-3, all other holes had flows out of the

collar of the hole at the time the holes were completed. Hole V-3 was a shallow hole which extended only 12 feet before reaching bedrock contact.

With time, the angle holes which extended furthest in the upstream direction, maintained water to El. 573, while two vertical holes which intersected bedrock about 18 feet upstream of the toe of the dam exhibited a diminishing water level. Hole V-3 maintained a relatively constant level, with water observed slightly above El. 572.

Headwater and tailwater elevations were read daily along with the observation hole readings. There appeared to be no apparent direct correlation between reservoir elevation and elevation of the water surface in the holes.

The drilled holes served another important function in that they established the elevation of bedrock at the base of the structure at each location. It was found that the average base elevation of the deep portion of the structure was approximately at El. 547, disregarding information from hole V-3, which was obviously high locally.

From the above observations it was determined that it would be reasonable to assume that the most representative base elevation for making the stability analysis would be El. 547. Also, that a safe and conservative value of piezometric head to be used in uplift pressure determination would be to El. 573 at both the angle and vertical hole intersections at the base of the structures.

Included in the appendix of this report are:

1. Drillers logs of drill holes - Plates III through XI
2. Water Level Observations - Plate XII

STABILITY ANALYSIS

The analysis was made to include the loading conditions which were established for the non-overflow section. These can be summarized as follows:

Case 1 - Normal Water Surface Condition

Headwater El. 663.3

Tailwater El. 563.3

Uplift

Case 2 - Design Flood Condition (20,000 cfs Flood)

Headwater El. 665.3

Tailwater El. 565.3

Uplift

Case 3 - Maximum Probable Flood Condition (100,000 cfs Flood)

Headwater El. 675.5

Tailwater El. 573.0

Uplift

Case 4 - Seismic Conditions

Headwater El. 663.3

Tailwater El. 563.3

Earthquake Forces 0.05g Horizontal Direction

Uplift

Uplift forces in all cases were based on the assumption of full headwater pressure at the upstream face of the structure varying linearly to the observed pressures at the intercept of the two lines of observation holes, and then varying linearly from the line of vertical holes to tail-

water pressure at the downstream face of the structure.

In calculating factors of safety against sliding, it was obvious that in many cases that the sliding factor exceeded the allowable value of friction for concrete to rock. It was therefore necessary to employ the shear value in the rock and to then check the combined shear - friction factor of safety provided.

Results of stability analyses for all cases considered are tabulated on Plate I in the appendix. A tabulation of values and assumptions used in the stability analysis are included on Plate II in the appendix.

CONCLUSIONS

Based on the results of the uplift observations and the ensuing stability calculations it is concluded that the non-overflow structure at Inghams Dam as analyzed satisfies the values of the safety factors prescribed as minimums in all cases. Inspection of the factor of safety against overturning in Case IV as calculated is so close numerically to 1.25 that for all practical purposes it was judged satisfactory.

Since the uplift forces influence to a great degree the numerical factors of safety as calculated, we should like to point out that based on the piezometric readings in the observation holes in the structure, it appears that the uplift pressure diagram used still provides a somewhat larger factor of safety than is apparent. This results from the assumption of full headwater pressure being applied at the upstream face of the structure and then varying linearly to the observed pressure at the first line of holes.

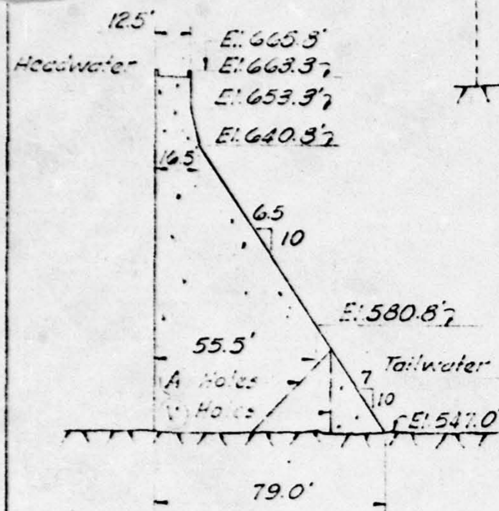
Although no observation holes were provided at or near the upstream face, it would seem unlikely that the loss in head between this face and the first line of holes would be in the order of 90 feet (El. 663 - El. 573 = 90) in a horizontal distance of 35 feet. Then remain, as observed at a constant

value for the next 26 feet, at all three transverse lines of observation holes. It seems more reasonable to assume that in actuality full headwater pressure is not present at the line of the upstream face. Should this be the case, then the uplift effect would thereby be reduced in all cases, resulting in improved factors of safety over those tabulated on Plate I.

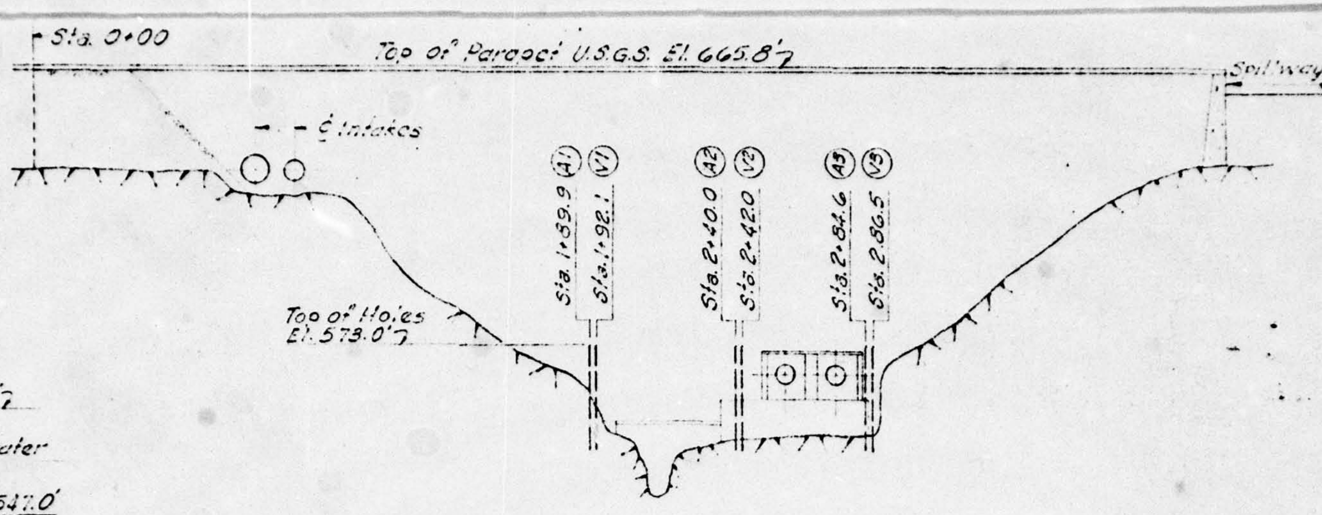
As a matter of interest, a comparison of the effect of the uplift as used in the calculations and the conventional assumption of full headwater at the heel varying to full tailwater at the toe was made. Headwater and tailwater elevations were taken as observed on June 19, 1970 as El. 662.5 and El. 559.7, respectively.

A comparison of the vertical effect showed that observed forces were 69.2 percent of the conventional assumption. This indicates that a net effect at this project would be the equivalent of the conventional uplift gradient of full headwater to tailwater applied over two thirds of the base area versus 100 percent of the area as normally considered at structures with no drainage provisions at the foundation.

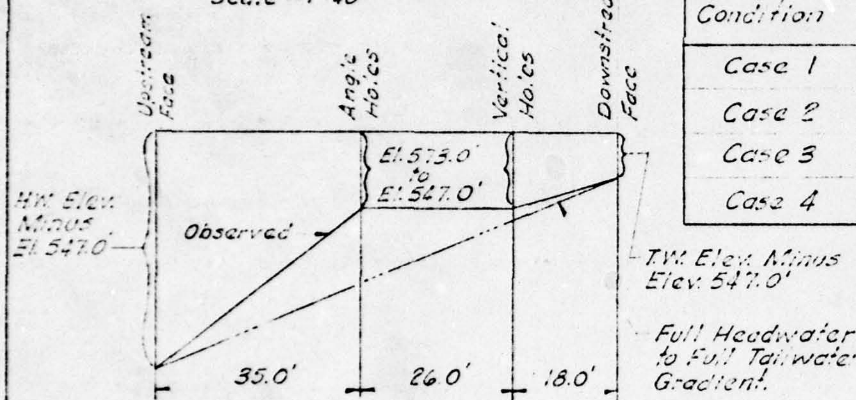
A comparison of the overturning effect about the toe was in about the same range, with the observed effect being about 73.5 percent of the conventional assumption.



DAM SECTION
Scale ~ 1":40'



DOWNSTREAM ELEVATION OF NONOVERFLOW DAM
Scale ~ 1":40'



UPLIFT PRESSURE DIAGRAM - HEAD IN FEET
Not to Scale

DAM STABILITY SUMMARY

Condition	Elev.	ΣV kips	ΣH kips	ΣH/ΣV	$S_{s.f}$	Resultant From Toe	ΣMr Ft. kips	ΣMo Ft. kips	ΣMr/ΣMo	Comp. Stress F.S.: Heel	Toe
Case 1	547.0	495.7	415.7	0.84	5.75	19.6	37867	28130	1.35	-22	110
Case 2	547.0	494.1	427.5	0.87	5.60	17.8	37894	29117	1.30	-28	114
Case 3	547.0	485.8	490.9	1.03	4.88	7.0	38073	34693	1.10	-63	145
Case 4	547.0	495.7	475.2	0.96	5.04	14.5	37867	30683	1.24	-39	127

CASE 1 - NORMAL WATER SURFACE CONDITION HEADWATER EL. 663.3 TAILWATER EL. 563.3, UPLIFT
CASE 2 - DESIGN FLOOD CONDITION (23,000 cfs FLOOD) " 665.9 " " 565.3, UPLIFT
CASE 3 - MAX. PROBABLE FLOOD " (100,000 cfs FLOOD) " 675.5 " " 573.3, UPLIFT
CASE 4 - SEISMIC CONDITION, EARTHQUAKE FORCE 0.05g HORIZONTAL " 663.3 " " 563.3, UPLIFT

NIAGARA MOHAWK POWER CORPORATION
SYRACUSE, N.Y.

INGHAMS DEVELOPMENT
EAST CANADA PROJECT NO. 2648
STABILITY ANALYSIS TABULATION
UHL, HALL & RICH ENGINEERS BOSTON, MASS.

NOVEMBER, 1970

PLATE I

TABULATION OF VALUES & ASSUMPTIONS

USED IN STABILITY ANALYSIS

PLATE II

1. Unit weight of concrete - 150 lb/cu. ft,
2. Unit weight of water - 62.4 lb/cu. ft.
3. Uplift in accordance with pressure diagram - Plate I
4. Earthquake acceleration - 0.05g
5. Shear - friction factor of safety - S_{s-f}

$$*S_{s-f} = \frac{f\Sigma(v) + r\, S_a A}{\Sigma H}$$

Where $r = 0.5$

$S_a = 380$ p.s.i.

A = Area of base

$f = 0.5$

ΣV = Summation of vertical forces

ΣH = Summation of horizontal forces

* For discussion and explanation of terms see <u>Hydroelectric Handbook</u> by Creager and Justin, Wiley & Sons, Inc.

Plate III

SOILTESTING, INC. 120 Mountain Road Seymour, Connecticut	CLIENT: NIAGARA MOHAWK POWER CORP. P.O. #23138	SHEET 1 OF 1 HOLE NO. V-1
FOREMAN – DRILLER DH - AM - RS	PROJECT NO B-742	BORING LOCATIONS as instructed by Engineer
INSPECTOR	PROJECT NAME INGHAMS HYDRO NON-OVERFLOW DAM	
	LOCATION TOWN OF MANHEIM HERKIMER CO., NEW YORK	OFFSET

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR	
AT ______ FT AFTER ______ HOURS	TYPE			NXM	Date Start 6/17/70 Date Fin 6/19/70
AT ______ FT AFTER ______ HOURS	SIZE I D				SURFACE ELEV 573.0'
	HAMMER WT			BIT	GROUND WATER ELEV *see note
	HAMMER FALL			Diamond	

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
										8		Run #1	Gray Concrete
		1	C	24"	23"	2'0"				11		2'0"	solid from face of Dam.
										10		Run #2	
		2	C	24"	25"	4'0"				10		4'0"	
										11		Run #3	
5		3	C	24"	24"	6'0"				11		6'0"	
										12		Run	
										14		#4	
										15			
										15			
10		4	C	60"	60"	11'0"				17		11'0"	
										17		Run	(Dolomite seam from 12'0" to
										15		#5	12'8")
										18			
										16			
15		5	C	60"	60"	16'0"				17		16'0"	
										17		Run	(Dolomite seam from 16'5" to
										18		#6	17'5")
										20			
										20			
20		6	C	60"	60"	21'0"				25		21'0"	(Dolomite seam from 21'3" to
		7	C	12"	12"	22'0"				45		Run #7	22'5")
										30		22'0"	22'9" - Concrete
										28		Run #8	half Concrete; half Dolomite
		8	C	36"	36"	25'0"				29		25'0"	vertiel seam to 24'3"
25										25		Run	Concrete seam to 25'8"
										27		#9	Gray/White Dolomite (fractured
										27			
										28			
		9	C	60"	60"	30'0"				28		30'0"	
30													End of Boring - 30'0"
35													

*Note: Hole filled with water on completion--water level stable for 2 hours.

GROUND SURFACE TO ______ FT. USED ______" CASING THEN ______" CASING TO ______ FT — HOLE NO. V-1

P: PIT A: AUGER UP: UNDISTURBED PISTON — C = COARSE

T: THINWALL V: VANE TEST — M = MEDIUM

S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER — F = FINE

TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

SOILTESTING, INC. 120 Mountain Road Seymour, Connecticut	CLIENT: NIAGARA MOHAWK POWER CORP. P.O. #23138	SHEET 1 OF 1 HOLE NO. V-2
	PROJECT NO B-742	BORING LOCATIONS as instructed by Engineer
FOREMAN —DRILLER DH - AM - RS	PROJECT NAME INGHAMS HYDRO NON-OVERFLOW DAM	
INSPECTOR	LOCATION TOWN OF MANHEIM HERKIMER CO., NEW YORK	OFFSET

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR	
AT ______ FT AFTER ______ HOURS	TYPE			NXM	Date St. 6/11/70 Date Fin. 6/12/70
AT ______ FT. AFTER ______ HOURS	SIZE I D				SURFACE ELEV 573.0'
	HAMMER WT			BIT	GROUND WATER ELEV *see note
	HAMMER FALL			Diamond	

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
										7		Run #1	Gray Concrete
		1	C	18"	12"	1'6"				8		1'6"	(first 18" shattered)
										8		Run #2	
		2	C	24"	24"	3'6"				10		3'6"	
5										10		Run	
										11		#3	
										11			
		3	C	60"	60"	8'6"				11		8'6"	
										11		Run	
10										10		#4	
										10			
										9			
		4	C	60"	60"	13'6"				9		13'6"	
										10		Run	
15										11		#5	
										11			
										10			
		5	C	60"	60"	18'6"				11		18'6"	
										12		Run	
20										11		#6	
										12			
										12			
		6	C	60"	60"	23'6"				12		23'6"	
										12		Run	
25										11		#7	
										14			25'10" - Top of Rock
										15			Gray/White Dolomite -
										15			fractured
		7	C	60"	60"	28'6"				18		28'6"	
30													End of Boring - 28'6"
35													
40													

*Note: filled hole with water at completion - no elevation change in 3 hours.

GROUND SURFACE TO ______ FT. USED ______" CASING THEN ______" CASING TO ______ FT

HOLE NO. V-2

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON C = COARSE

UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST M = MEDIUM

O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER F = FINE

PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

Plate V

SOILTESTING, INC.
120 Mountain Road
Seymour, Connecticut

CLIENT: NIAGARA MOHAWK POWER CORP.
P.O. #23138

SHEET 1 OF 1
HOLE NO. V-2
Phase 2

PROJECT NO: B-742

BORING LOCATIONS: as instructed by Engineer

FOREMAN - DRILLER: DD - AM - RS

PROJECT NAME: INGHAMS HYDRO NON-OVERFLOW DAM

INSPECTOR:

LOCATION: TOWN OF MANHEIM
HERKIMER CO., NEW YORK

OFFSET:

GROUND WATER OBSERVATIONS
AT ____ FT AFTER ____ HOURS
AT ____ FT AFTER ____ HOURS

	CASING	SAMPLER	CORE BAR
TYPE			NXM
SIZE I D			
HAMMER WT			BIT
HAMMER FALL			Diamond

Date Start 6/17/70 Date Fin 6/17/70
SURFACE ELEV 573.0'
GROUND WATER ELEV ____

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
													Engineer requested drilling an additional 2 feet.
5													
10													
15													
20													
25													
												28'6"	start Phase 2
										18		Run #1	Gray/White Dolomite with
30		1	C	24"	24"	30'6"				20		30'6"	vertical and horizontal seams.
													End of Boring - 30'6"
35													
40													

GROUND SURFACE TO ____ FT. USED ____" CASING THEN ____" CASING TO ____ FT

HOLE NO. V-2
Phase 2

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON
UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST
O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER
PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

C = COARSE
M = MEDIUM
F = FINE

Plate VI

SOILTESTING, INC.
120 Mountain Road
Seymour, Connecticut

CLIENT: NIAGARA MOHAWK POWER CORP.
P.O. #23138

SHEET 1 OF 1
HOLE NO. V-3

PROJECT NO: B-742

BORING LOCATIONS: as instructed by Engineer

FOREMAN - DRILLER: DH - AM - RS

INSPECTOR:

PROJECT NAME: INGHAMS HYDRO NON-OVERFLOW DAM

LOCATION: TOWN OF MANHEIM
HERKIMER CO., NEW YORK

OFFSET:

GROUND WATER OBSERVATIONS
AT ____ FT AFTER ____ HOURS
AT ____ FT AFTER ____ HOURS

	CASING	SAMPLER	CORE BAR
TYPE			NXM
SIZE I D			
HAMMER WT			BIT
HAMMER FALL			Diamond

Date Start 6/4/70 Date Fin. 6/5/70
SURFACE ELEV 573.0'
GROUND WATER ELEV *see note

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
										7		Run #1	Concrete
		1	C	24"	24"	2'0"				7		2'0"	
										7		Run #2	
		2	C	24"	24"	4'0"				8		4'0"	Concrete
5										9		Run	
										10		#3	Concrete; trace of Dolomite.
										10			
										13			
		3	C	60"	60"	9'0"				11		9'0"	
										12		Run	
10										11		#4	
										11			12'0" - Top of Rock
										15			Gray/White Shale and Dolomite,
		4	C	60"	60"	14'0"				15		14'0"	Limestone.
15													End of Boring = 14'0"
20													
25													*Note: Filled hole with water on completion-water down 21" on June 6.
30													
35													
40													

GROUND SURFACE TO ____ FT. USED ____" CASING THEN ____" CASING TO ____ FT

HOLE NO. V-3

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON
UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST
O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER
PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

C = COARSE
M = MEDIUM
F = FINE

Plate VII

SOILTESTING, INC.
120 Mountain Road
Seymour, Connecticut

CLIENT NIAGARA MOHAWK POWER CORP.
P.O. #23138

SHEET 1 OF 1
HOLE NO. V-3
Phase 2

PROJECT NO B-742

BORING LOCATIONS as instructed by Engineer

FOREMAN - DRILLER DH - AM - RS

PROJECT NAME INGHAMS HYDRO NON-OVERFLOW DAM

INSPECTOR

LOCATION TOWN OF MANHEIM
HERKIMER CO., NEW YORK

OFFSET

GROUND WATER OBSERVATIONS
AT ______ FT AFTER ______ HOURS
AT ______ FT AFTER ______ HOURS

	CASING	SAMPLER	CORE BAR
TYPE			NXM
SIZE I D			
HAMMER WT			BIT
HAMMER FALL			Diamond

Date Start 6/10/70 Date Fin. 6/10/70
SURFACE ELEV 573.0'
GROUND WATER ELEV *see note

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST / MOIST	STRATA CHANGE DEPTH / ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
													Moved back on boring at request of Engineer.
												14'0"	START Phase 2
15										15		Run	Gray/white Dolomite-
										12		#1	15'0" Limestone.
										11			Concrete
										11			
		1	C	60"	60"	19'0"				11		19'0"	
20										12		Run	
										13		#2	
										14			
										14			
		2	C	54"	54"	23'6"				15		23'6"	
25										18		Run	
										17		#3	Gray/white Dolomite.
										17			
										16			
		3	C	60"	60"	28'6"				17		28'6"	
30													End of Boring - 28'6"
35													*Note: water running out of boring, very slow.
40													

GROUND SURFACE TO ______ FT. USED ______" CASING THEN ______" CASING TO ______ FT

HOLE NO. V-3
Phase 2

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON
UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST
O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER
PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

C = COARSE
M = MEDIUM
F = FINE

SOILTESTING, INC.
120 Mountain Road
Seymour, Connecticut

CLIENT: NIAGARA MOHAWK POWER CORP.
P.O. #23138

SHEET 1 OF 1
HOLE NO. A-1

FOREMAN - DRILLER: DH - AM - RS

INSPECTOR:

PROJECT NO: B-742

PROJECT NAME: INGHAMS HYDRO NON-OVERFLOW DAM

LOCATION: TOWN OF MANHEIM, HERKIMER CO., NEW YORK

BORING LOCATIONS: as instructed by Engineer

OFFSET:

GROUND WATER OBSERVATIONS
AT ____ FT AFTER ____ HOURS
AT ____ FT AFTER ____ HOURS

	CASING	SAMPLER	CORE BAR
TYPE			NXM
SIZE I D			
HAMMER WT			BIT
HAMMER FALL			Diamond

Date Start 6/19/70 Date Fin. 6/23/70
SURFACE ELEV 573.0'
GROUND WATER ELEV *see note

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
										10		Run #1	Gray Concrete
		1	C	24"	22"	2'0"				9		2'0"	
										10		Run #2	
		2	C	24"	22"	4'0"				10		4'0"	
										12		Run #3	5'2"
5										15			Layer Dolomite (from 5'2" to
										18			7'8" 7'8")
										16			
		3	C	60"	62"	9'0"				14		9'0"	Concrete(from 7'8" to 12'0")
										14		Run	
10										14		#4	
										13			12'0"
										17			Gray/White Dolomite
		4	C	60"	62"	14'0"				19		14'0"	(from 12'0" to 14'0")
										14		Run	
15										12		#5	Concrete
										13			(from 14'0" to 34'6")
										13			
		5	C	60"	60"	19'0"				14		19'0"	
										12		Run	
20										14		#6	
										12			
										13			
		6	C	60"	60"	24'0"				13		24'0"	
										14		Run	
25										13		#7	
										14			
										14			
		7	C	60"	60"	29'0"				14		29'0"	
30										13		Run	
										13		#8	
										14			
										15			
		8	C	60"	60"	34'0"				14		34'0"	
										17		Run	34'6"
35										18		#9	Gray/White Dolomite-fractured
										19			
		9	C	48"	48"	38'0"				20		38'0"	
													End of Boring - 38'0"
40													

*Note: Filled hole -water stable in boring--no loss or gain.

GROUND SURFACE TO ____ FT. USED ____" CASING THEN ____" CASING TO ____ FT

HOLE NO. A-1

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON
UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST
O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER
PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

C = COARSE
M = MEDIUM
F = FINE

Plate IX

SOILTESTING, INC.
120 Mountain Road
Seymour, Connecticut

CLIENT: NIAGARA MOHAWK POWER CORP.
P.O. #23138

SHEET 1 OF 2
HOLE NO. A-2

PROJECT NO: B-742

BORING LOCATIONS: as instructed by Engineer

FOREMAN - DRILLER: DH - AM - RS

PROJECT NAME: INGHAMS HYDRO NON-OVERFLOW DAM

INSPECTOR:

LOCATION: TOWN OF MANHEIM, HERKIMER CO., NEW YORK

OFFSET:

GROUND WATER OBSERVATIONS
AT ______ FT AFTER ______ HOURS
AT ______ FT AFTER ______ HOURS

	CASING	SAMPLER	CORE BAR
TYPE			NXM
SIZE I D			
HAMMER WT			BIT
HAMMER FALL			Diamond

Date Start 6/12/70 Date Fin. 6/16/70
SURFACE ELEV 573.0'
GROUND WATER ELEV *see note

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		1	C	18"	13"	1'6"				7		Run #1	
										8		1'6"	Gray weathered concrete -
		2	C	18"	20"	3'0"				10		Run #2	top 12" to 18"
										18		3'0"	
5										22		Run #3	
										17			
		3	C	54"	54"	7'6"				19		7'6"	
										13		Run	Gray Concrete; some decayed
										14		#4	stone.
10										17			
										16			
		4	C	60"	55"	12'6"				16		12'6"	
										17		Run	
										25		#5	
15										23			
										27			
		5	C	60"	60"	17'6"				26		17'6"	
										26		Run #6	
		6	C	12"	12"	18'6"				28		18'6"	
20										25		Run	
										25		#7	
		7	C	48"	48"	22'6"				23		22'6"	
										18		Run	
										20		#8	
25										20			
										19			
		8	C	60"	57"	27'6"				21		27'6"	
										20		Run	
										21		#9	
30										19			
										20			
		9	C	60"	60"	32'6"				20		32'6"	
										21		Run	
										19		#10	
35										20			
										20			
		10	C	60"	60"	37'6"				22		37'6"	Concrete
										21		Run	
										20		#11	
40										26			

GROUND SURFACE TO ______ FT. USED ______" CASING THEN ______" CASING TO ______ FT

HOLE NO. A-2

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON
UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST
O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER
PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

C = COARSE
M = MEDIUM
F = FINE

Plate X

SOILTESTING, INC.
120 Mountain Road
Seymour, Connecticut

CLIENT: NIAGARA MOHAWK POWER CORP.
P.O. #23138

SHEET 2 OF 2
HOLE NO. A-2

PROJECT NO: B-742

BORING LOCATIONS: as instructed by Engineer

FOREMAN - DRILLER: DH - AM - RS

PROJECT NAME: INGHAMS HYDRO NON-OVERFLOW DAM

INSPECTOR:

LOCATION: TOWN OF MANHEIM
HERKIMER CO., NEW YORK

OFFSET:

GROUND WATER OBSERVATIONS
AT ______ FT AFTER ______ HOURS
AT ______ FT AFTER ______ HOURS

	CASING	SAMPLER	CORE BAR
TYPE			NXM
SIZE I.D.			
HAMMER WT			BIT
HAMMER FALL			Diamond

Date Start 6/12/70 Date Fin. 6/16/70
SURFACE ELEV 573.0'
GROUND WATER ELEV *see note

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
40		11	C	48"	41"	41'6"				8		Run #11	Gray concrete. Brown Sandy wash came out of voic between
										6		41'6"	40'6" and 41'6". Gray cement,
										15		Run #12	no stone between 41' and 41'4".
		12	C	24"	20"	43'6"				20		43'6"	Gray/White fractured Dolomite.
45													End of Boring - 43'6"
50													
55													Note: Water down to 15' at 4:30 PM on June 15, 1970. On June 16, 1970 water flowing out of hole before last 24" run; after run, water advanced up hole at 30"/min. or better.
60													
65													
70													
75													
80													

GROUND SURFACE TO ______ FT. USED ______" CASING THEN ______" CASING TO ______ FT

HOLE NO. A-2

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON C = COARSE
UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST
O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER M = MEDIUM
PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%. F = FINE

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

CLIENT: NIAGARA MOHAWK POWER CORP.
P.O. #23138

SHEET 1 OF 1
HOLE NO. A-3

PROJECT NO B-742

BORING LOCATIONS as instructed by Engineer

FOREMAN - DRILLER DH - AM - RS

PROJECT NAME INGHAMS HYDRO NON-OVERFLOW DAM

INSPECTOR

LOCATION TOWN OF MANHEIM HERKIMER CO., NEW YORK

OFFSET

GROUND WATER OBSERVATIONS
AT ____ FT AFTER ____ HOURS
AT ____ FT AFTER ____ HOURS

	CASING	SAMPLER	CORE BAR
TYPE			NXM
SIZE I D			
HAMMER WT			BIT
HAMMER FALL			Diamond

Date Start 6/5/70 Date Fin. 6/9/70
SURFACE ELEV 573.0'
GROUND WATER ELEV *see note

DEPTH	CASING BLOWS PER FOOT	SAMPLE NO	TYPE	PEN	REC	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE) 0-6	6-12	12-18	CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
										9		Run #1	
		1	C	24"	24"	2'0"				10		2'0"	Concrete - with large stone
										11		Run #2	aggregate
		2	C	24"	18"	4'0"				14		4'0"	
5										60		Run #3	
		3	C	18"	24"	5'6"				14		5'6"	
										12		Run	
										13		#4	
		4	C	48"	48"	9'6"				15		9'6"	
										14			
10										13		Run	
										13		#5	
		5	C	36"	36"	12'6"				13		12'6"	
										14		Run	
										16		#6	
15										15			
										15			
		6	C	60"	60"	17'6"				14		17'6"	
										14		Run	
										15		#7	
20										15			
										15			
		7	C	60"	60"	22'6"				16		22'6"	
										15		Run	
										14		#8	
25										15			
										15			
		8	C	60"	60"	27'6"				16		27'6"	
										16		Run	
										14		#9	
30										15			
										16			
		9	C	60"	60"	32'6"				16		32'6"	
										16		33'0"	Top of Rock
										15		Run	
35										17		#10	Gray/White Dolomite, Limeston
										19			fractured.
										20			
		10	C	60"	60"	37'6"				20		37'6"	
													End of Boring - 37'6"
40													

*Note: Making water; very slow stream; 6/9/70 after completion.

GROUND SURFACE TO ____ FT. USED ____" CASING THEN ____" CASING TO ____ FT

HOLE NO. A-3

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON
UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST
O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER
PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

C = COARSE
M = MEDIUM
F = FINE

NIAGARA MOHAWK POWER CORPORATION

INGHAMS DAM
NON-OVERFLOW SECTION

WATER LEVEL OBSERVATIONS

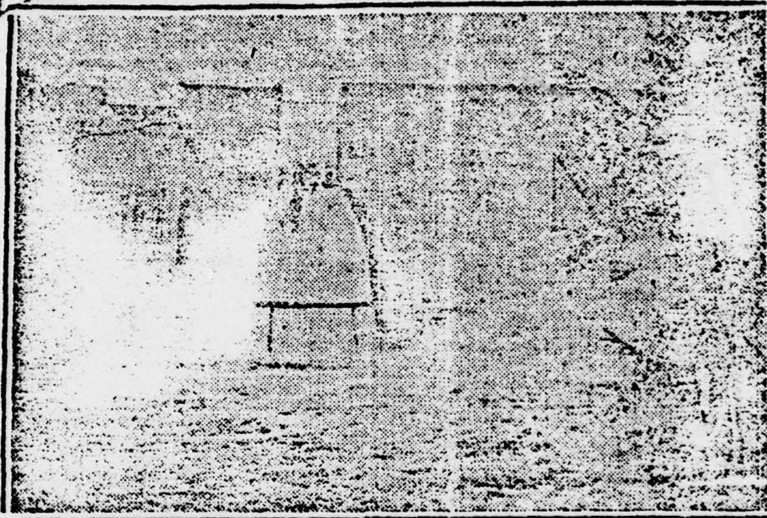
1970	H.W. ELEV.	T.W. ELEV.	AIR TEMP. °F.	ELEVATIONS OF WATER IN HOLES A-1	V-1	A-2	V-2	A-3	V-3
JUNE 12	657.0		68				573.0	573.0	572.29
JUNE 15	658.5		68				573.0	573.0	571.62
JUNE 16	657.3		68				573.0	573.0	571.33
JUNE 17	657.8		68			573.3	573.0	573.0	571.33
JUNE 18	658.0		68			573.3	570.08	573.0	572.25
JUNE 19	662.5	559.7	68		573.0	573.3	573.1	573.0	572.33
JUNE 22	660.3	555.5	57		573.0	573.3	570.75	573.0	572.33
JUNE 23	659.7	555.3	60		572.17	573.3	570.50	573.0	572.25
JUNE 24	656.3	553.3	60	573.0	571.83	573.3	570.00	573.0	572.17
JUNE 25	655.9	552.9	63	573.0	571.37	573.3	569.04	573.0	572.33
JUNE 26	657.7	553.4	67	573.0	570.92	573.3	569.50	573.0	572.17
JUNE 29	661.0	552.5	65	573.0	570.08	573.3	570.08	573.0	572.08
JUNE 30	661.1	553.4	64	573.0	569.83	573.3	569.83	573.0	572.33
JULY 1	661.0	553.5	68	573.0	569.58	573.3	570.33	573.0	572.25
JULY 2	659.0	553.2	68	573.0	569.42	573.3	569.92	573.0	572.33
JULY 7	660.7	553.1	72	573.0	568.67	573.3	569.92	573.0	572.21
JULY 13	660.7	553.1	75	573.0	568.29	573.3	569.67	573.0	572.08
JULY 21	662.0	559.7	74	573.0	568.33	573.3	573.0	573.0	572.29
JULY 27	660.9	553.1	85	573.0	568.08	573.3	571.33	573.0	572.17
AUGUST 4	661.3	553.0	82	573.0	568.67	573.3	570.83	573.0	572.29
AUGUST 11	659.3	552.8	80	573.0	568.33	573.3	570.50	573.0	572.08
AUGUST 18	659.9	553.0	72	573.0	568.25	573.3	570.62	573.0	572.17
SEPT. 1	658.8	552.9	70	573.0	570.08	573.3	571.25	573.0	572.21
SEPT. 9	660.5	553.0	61	573.0	569.83	573.3	571.50	573.0	572.25
SEPT. 15	661.2	553.0	52	573.0	570.00	573.3	571.75	573.0	572.21
SEPT. 21	661.6	553.0	69	573.0	570.17	573.3	572.00	573.0	572.17

APPENDIX G

DRAWINGS

Power Dam at Inghams Mills

Syracuse Hearld, Nov. 5, 1911.



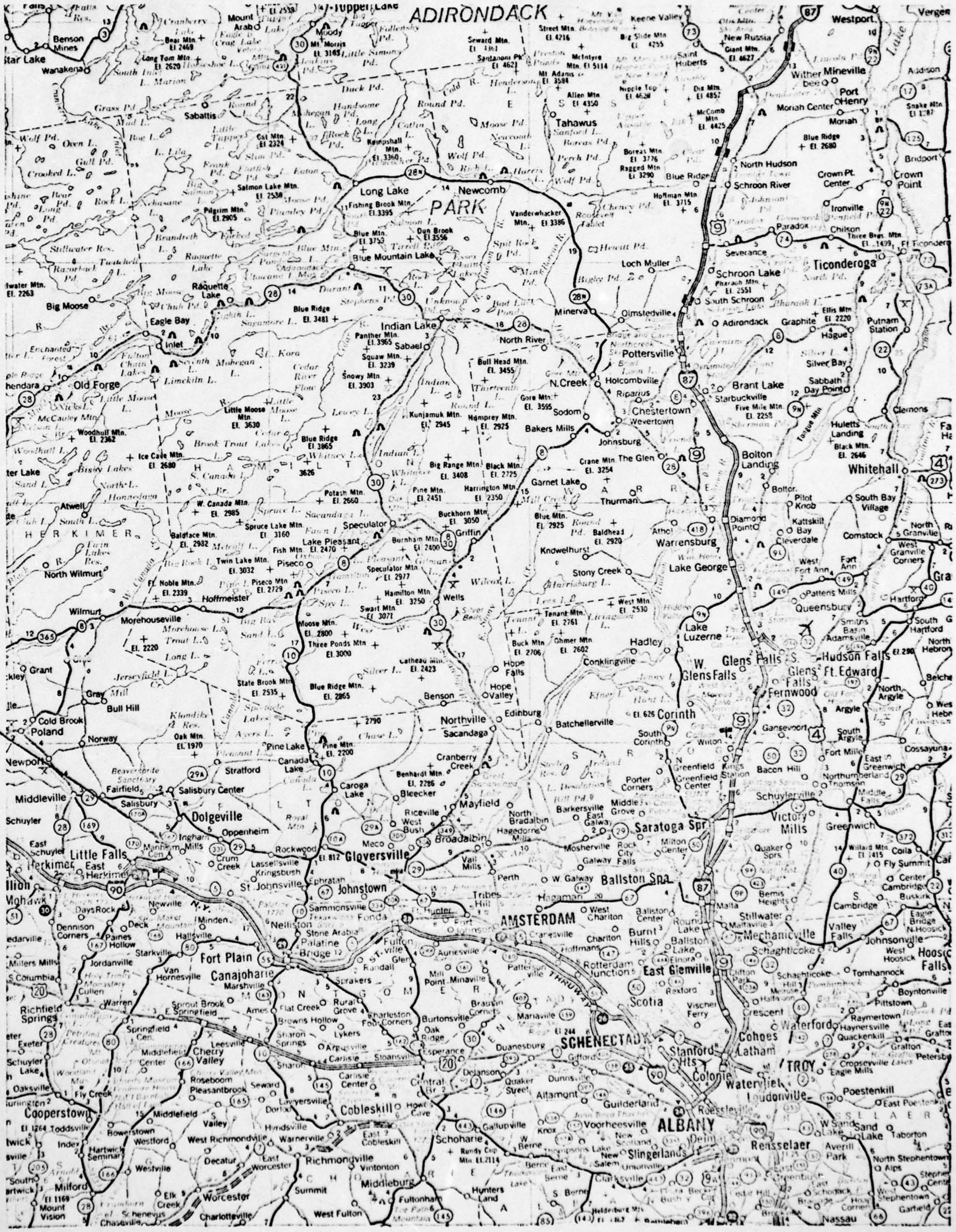
Little Falls, Nov. 4.—The bog dam in the East Canada creek at Inghams Mills is rapidly nearing completion and is attracting many visitors from all parts of the State. The structure is being erected by the East Creek Light and Power company and is three miles east of this city. When completed in about a month it will furnish power that will be sold to the Fonda, Johnstown & Gloversville Railroad company and to the villages in the Mohawk valley east of this city. The proposed Little Falls & Johnstown Railroad company will also use power from this plant.

The dam is said to be the highest of any in this part of the State and is built in a narrow gully in the East Canada creek. It will flood the valley back as far as Dolgeville. Already the dam is half filled and all that remains for its completion is the building of a small section in the center. The artificial lake that will be made by this large body of water promises to become a popular summer resort for Little Falls.

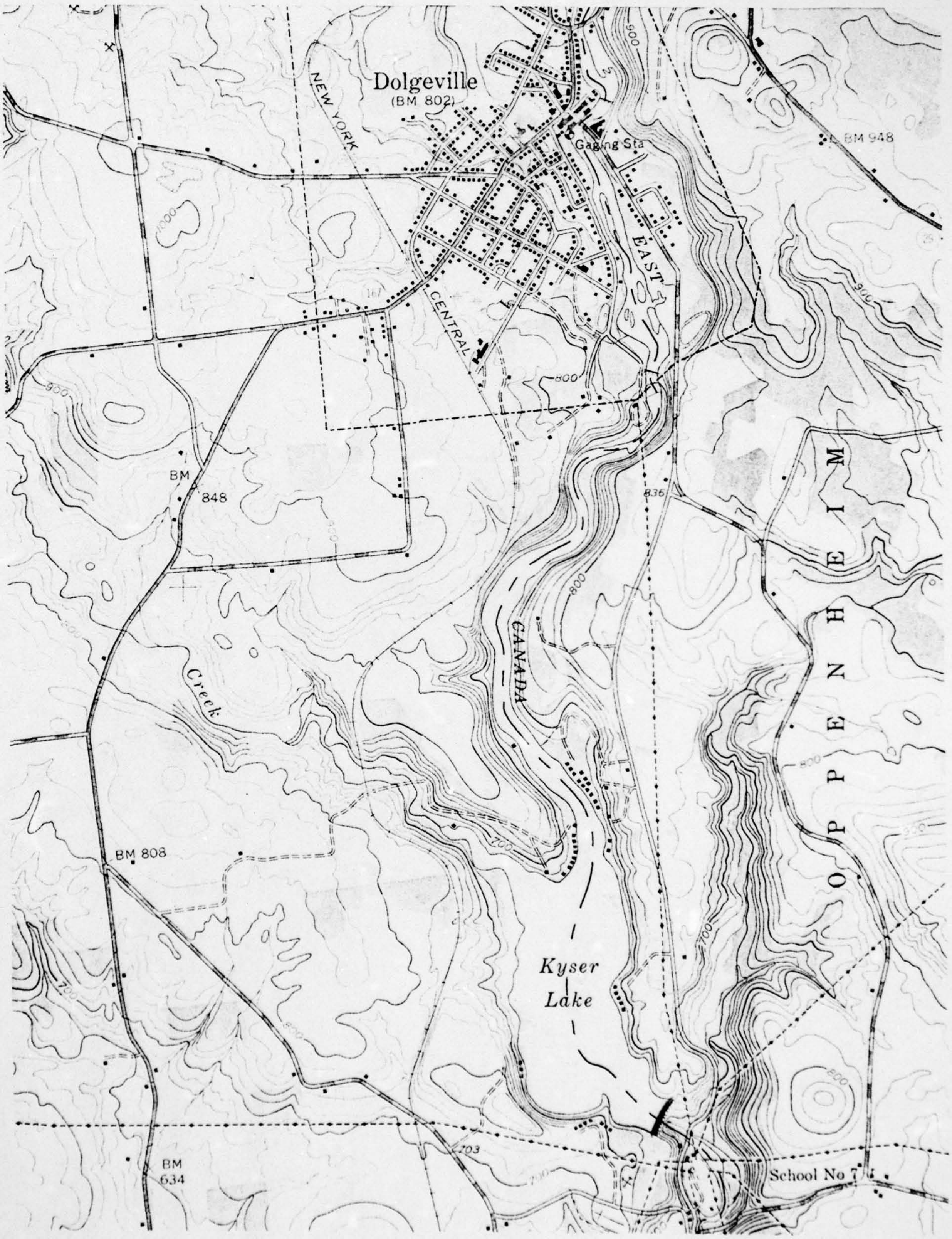
NEWSPAPER CLIPPING FROM SYRACUSE HEARLD

NOVEMBER 5, 1911

DAM



VICINITY MAP

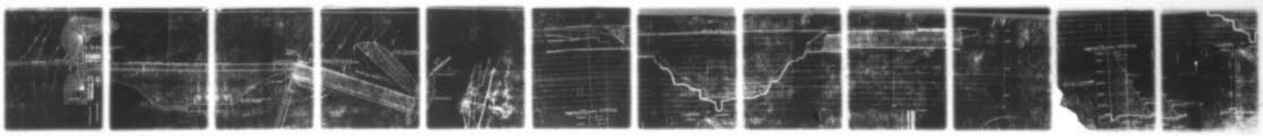


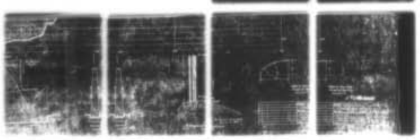
TOPOGRAPHIC MAP

AD-A077 423 NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/13
NATIONAL DAM SAFETY PROGRAM. INGHAMS DAM (INVENTORY NUMBER NY 1--ETC(U)
AUG 79 G KOCH DACW51-79-C-0001

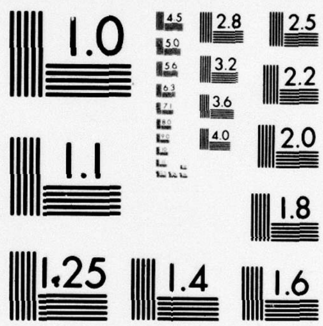
UNCLASSIFIED NL

2 OF 2
ADA
077423



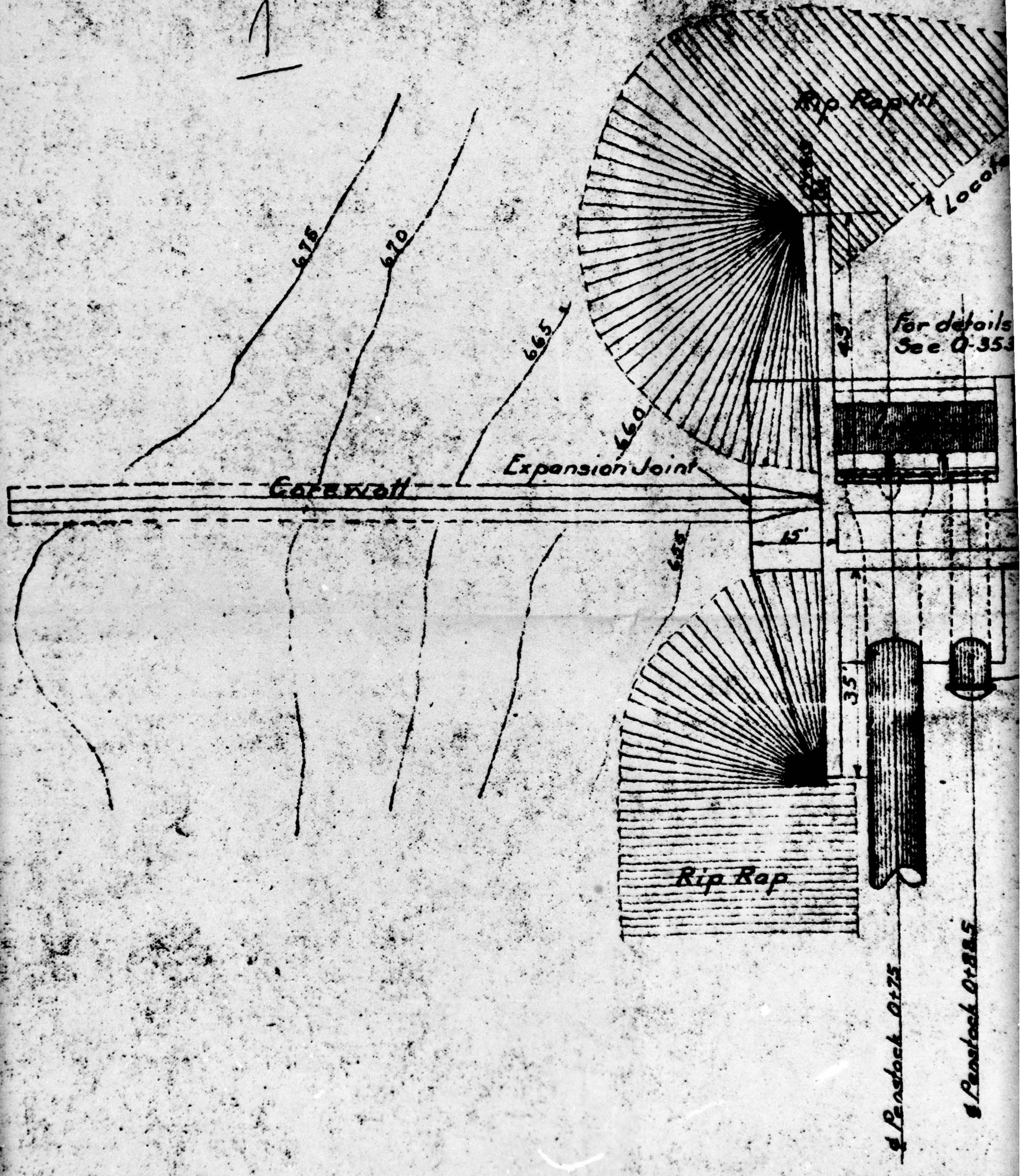


END
DATE
FILMED
1 -80
DDC

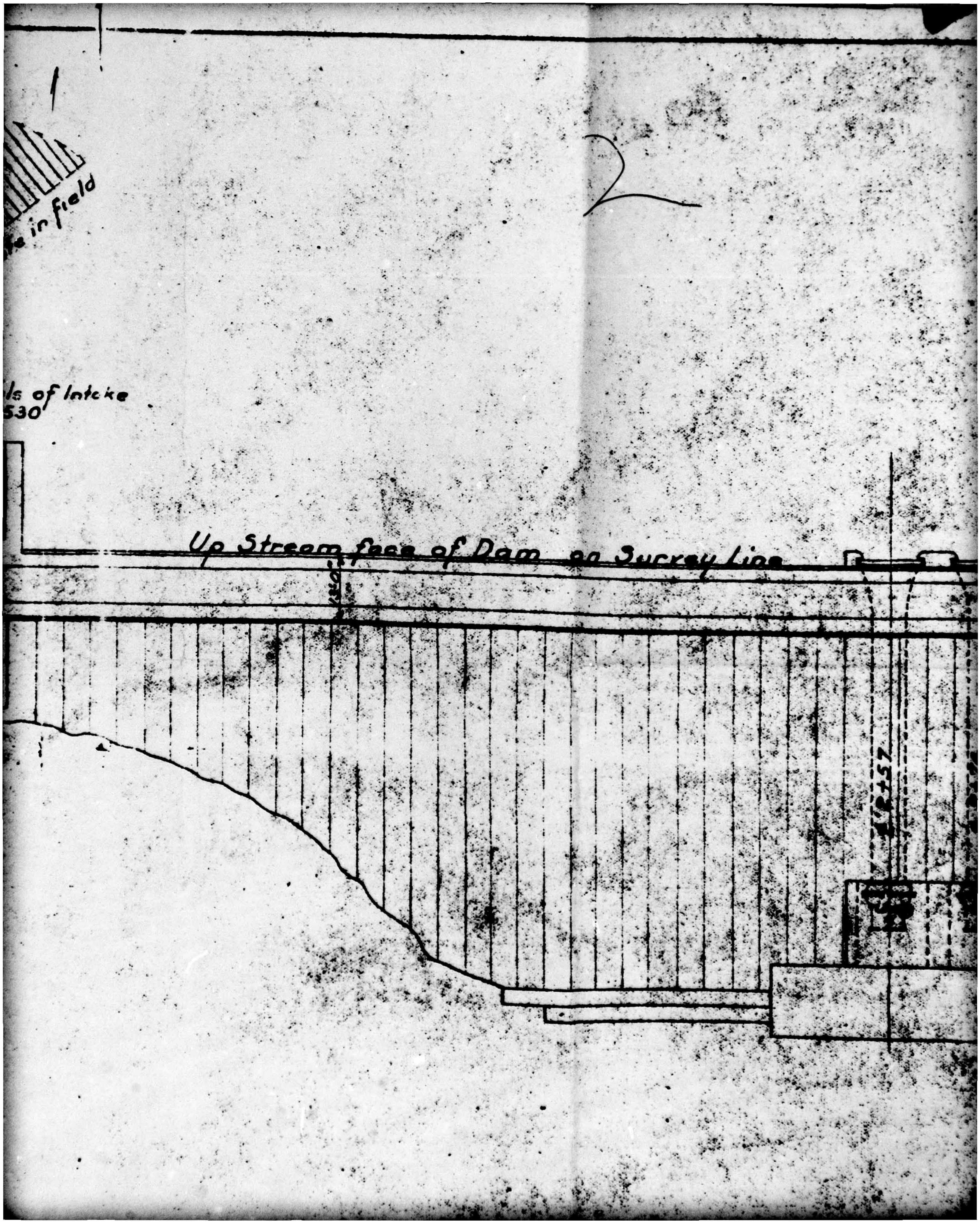


MICROCOPY RESOLUTION TEST CHART

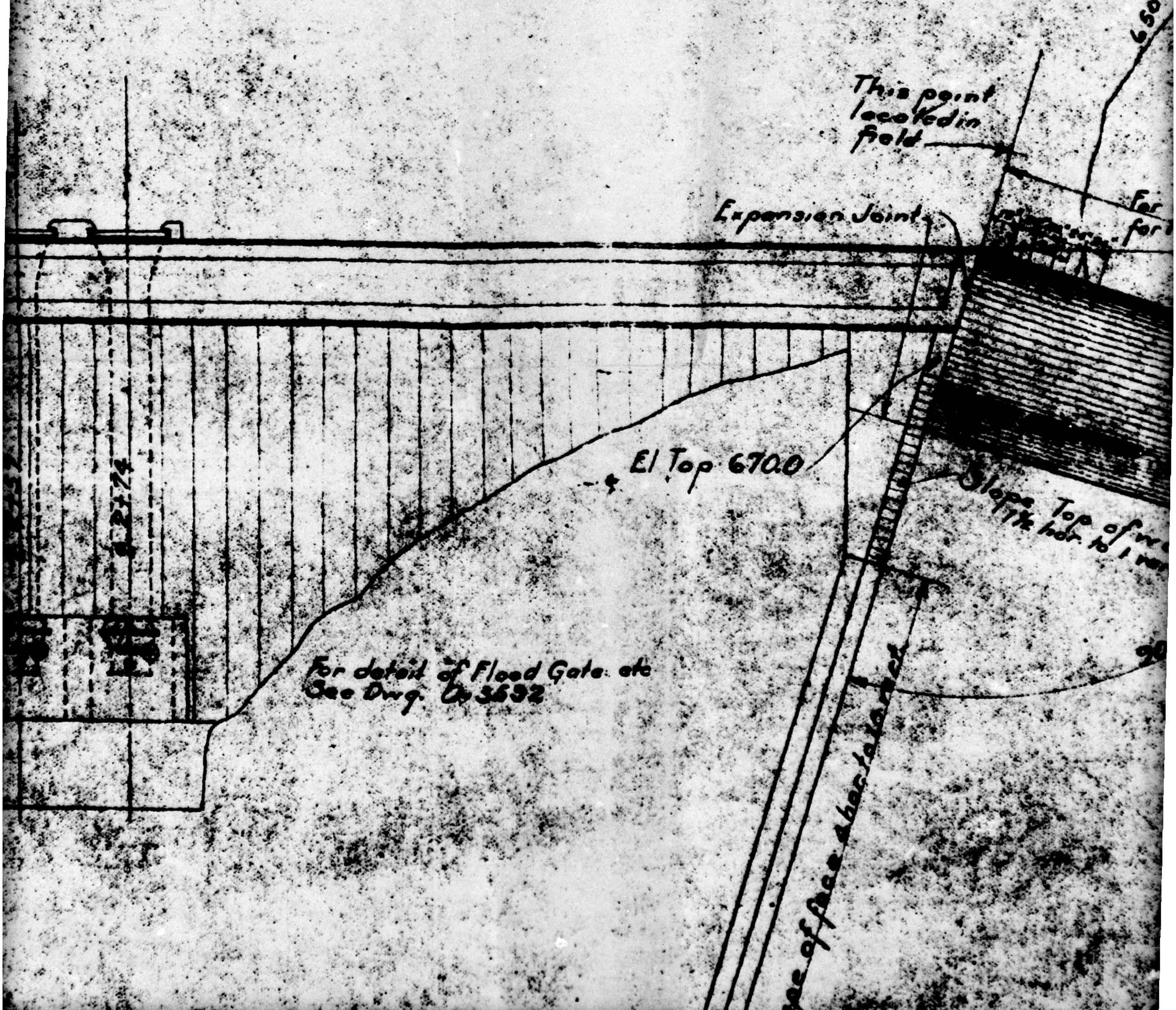
1



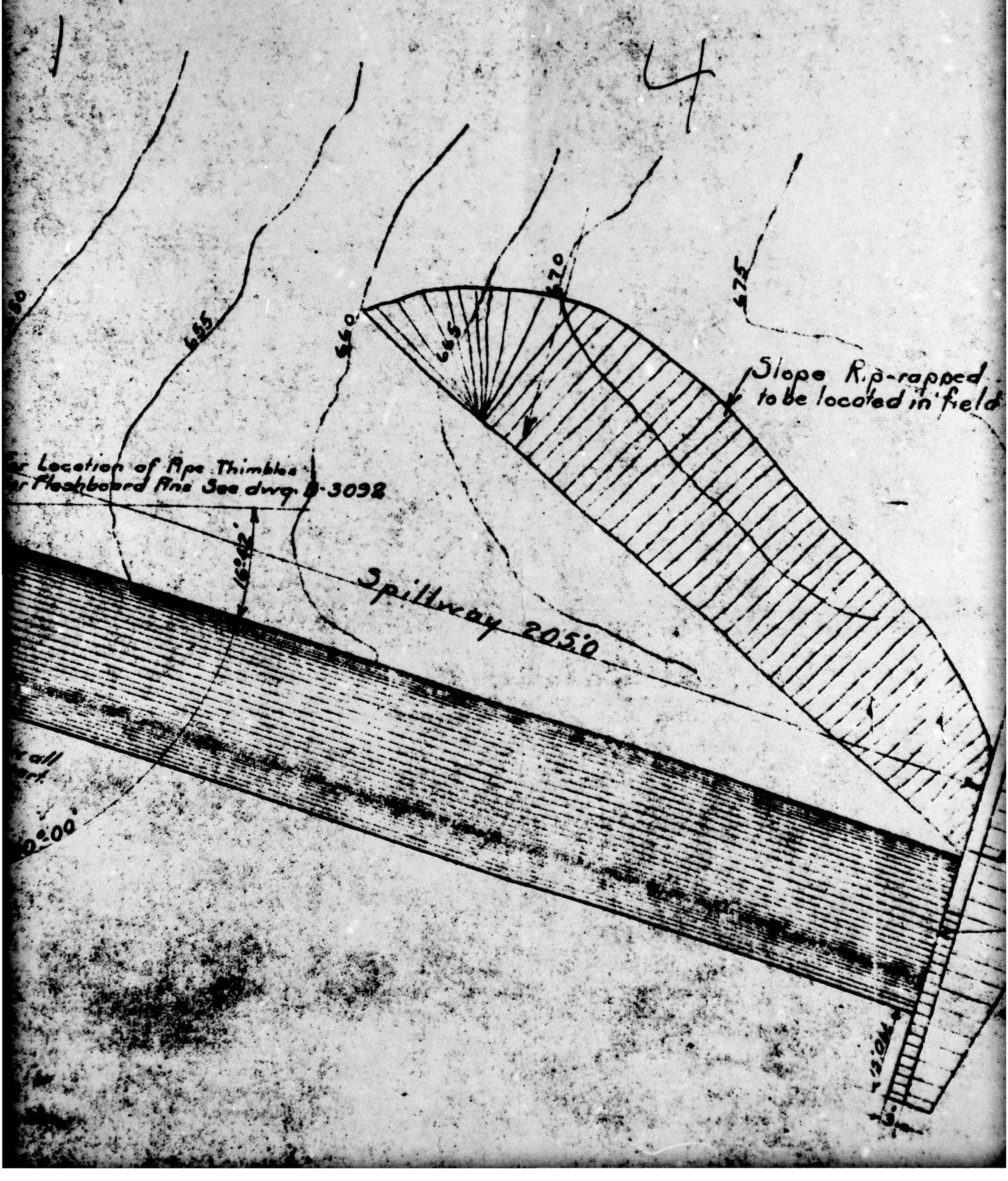
Rip Rap
Locat
675
670
665
660
655
For details
See O-355
Expansion Joint
Corewall
15'
35'
Rip Rap
℄ Penstock 0+75
℄ Penstock 0+88.5


in field
ls of Intake
530
Up Stream face of Dam on Survey Line

3



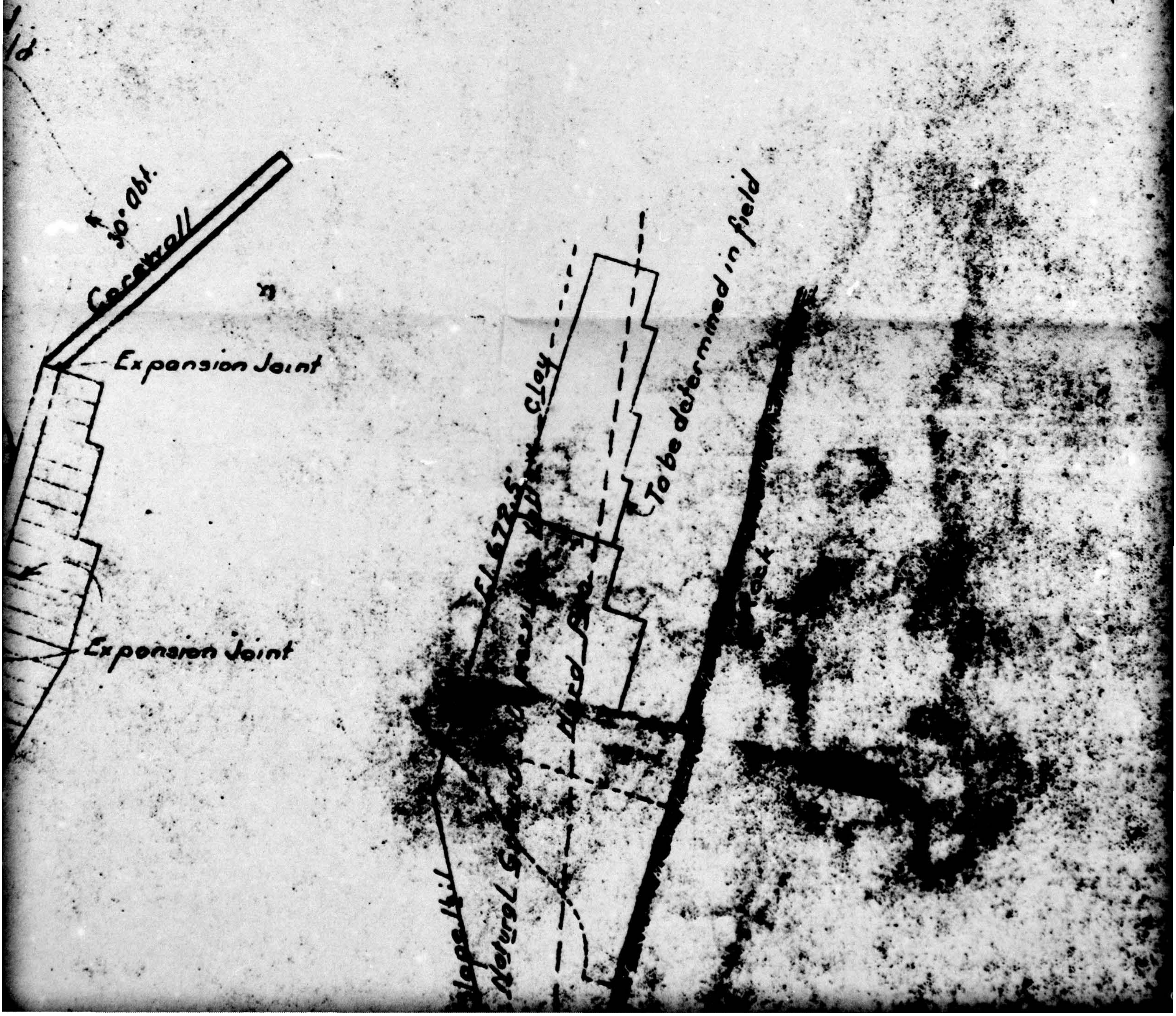
650
This point located in field
For
for
Expansion Joint
El Top 670.0
Slope Top of
1½ hor. to 1
90
For detail of Flood Gate etc
See Dwg. 3632
Slope of face abutment



1
4
655
660
665
670
675
Slope R.p-rapped
to be located in field
For Location of Pipe Thimbles
for Flashboard Pins See dwg. B-3092
16'-0"
Spillway 205.0

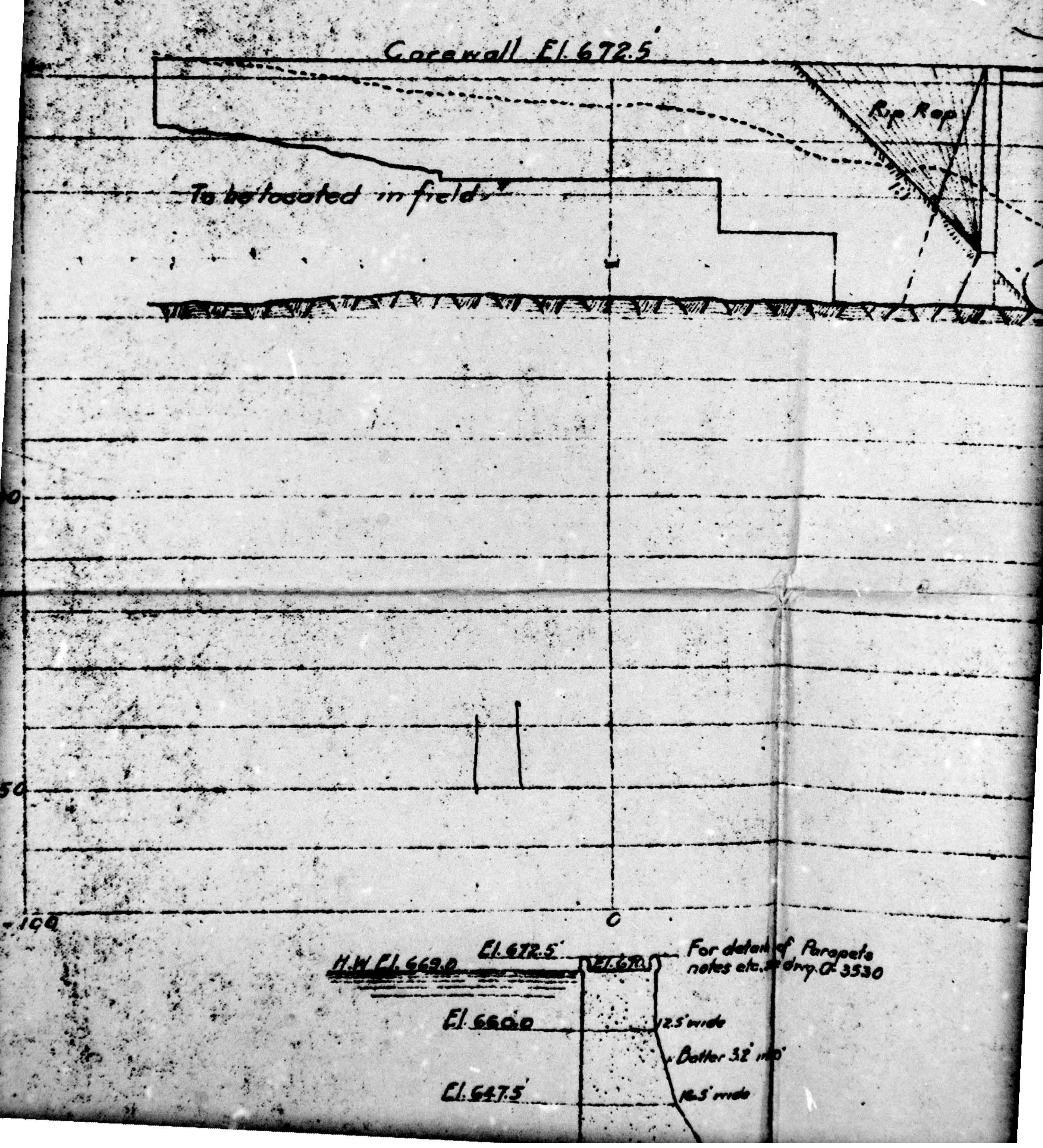
1

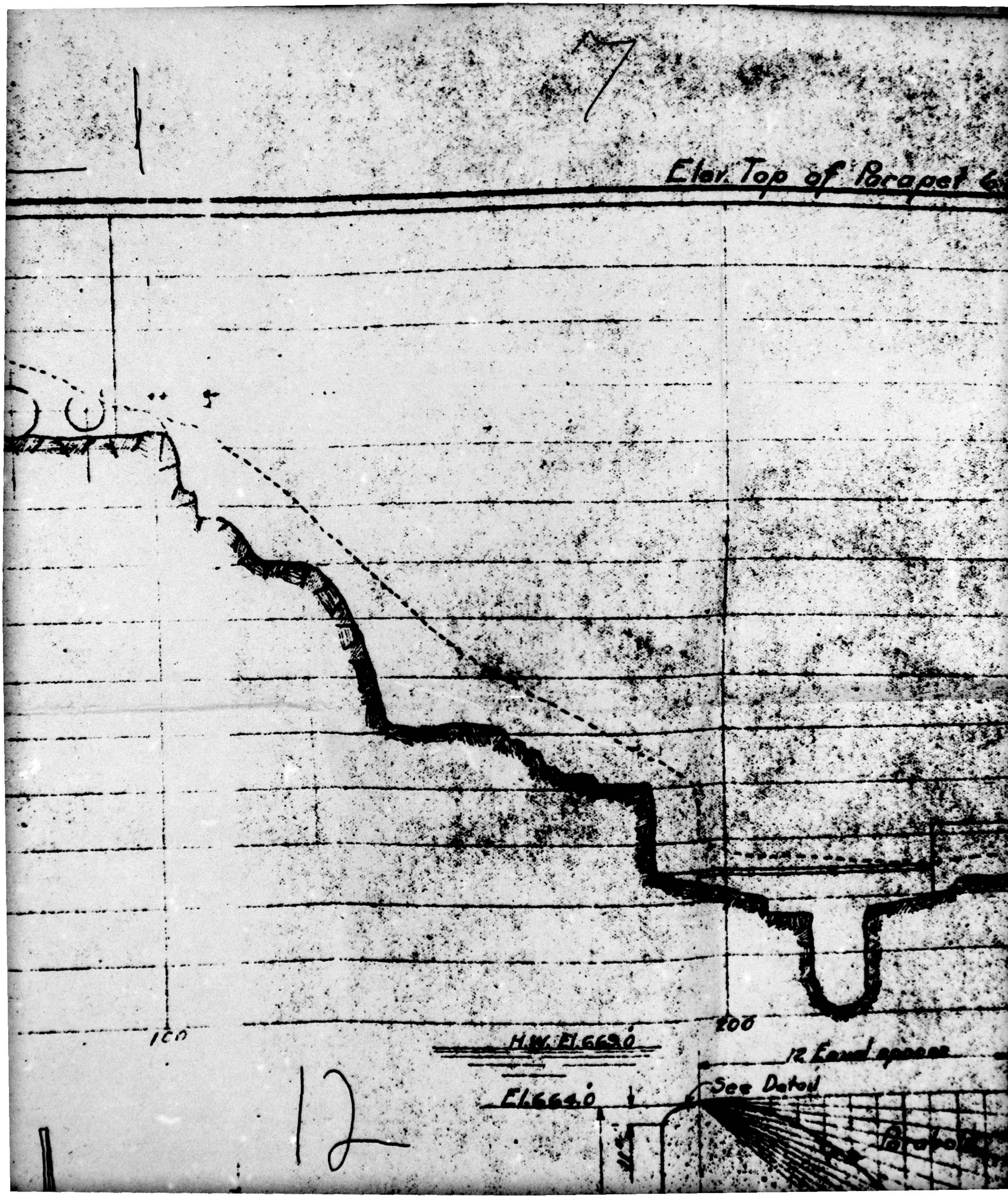
5



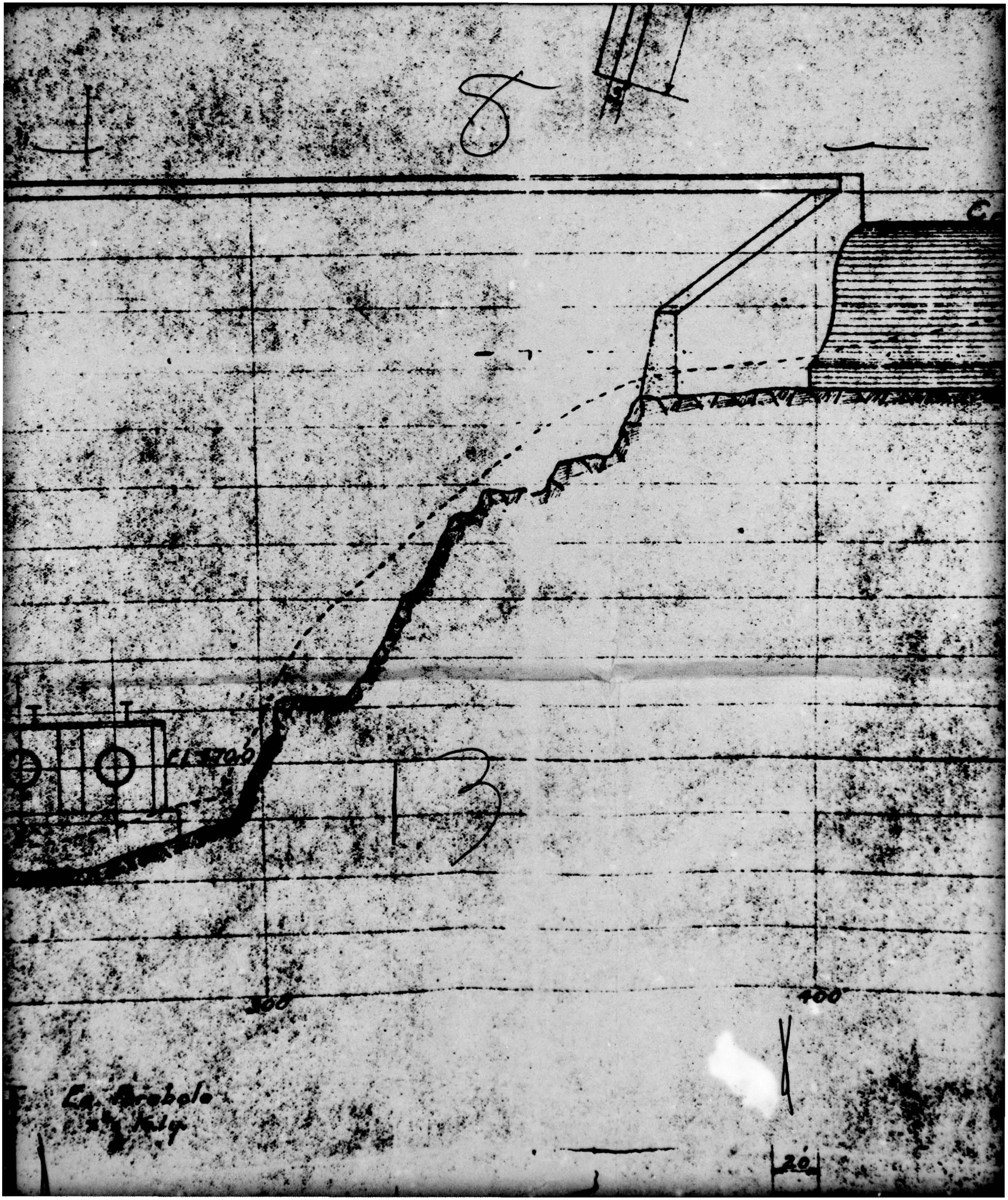
Corewall
Expansion Joint
Expansion Joint
Clay
To be determined in field

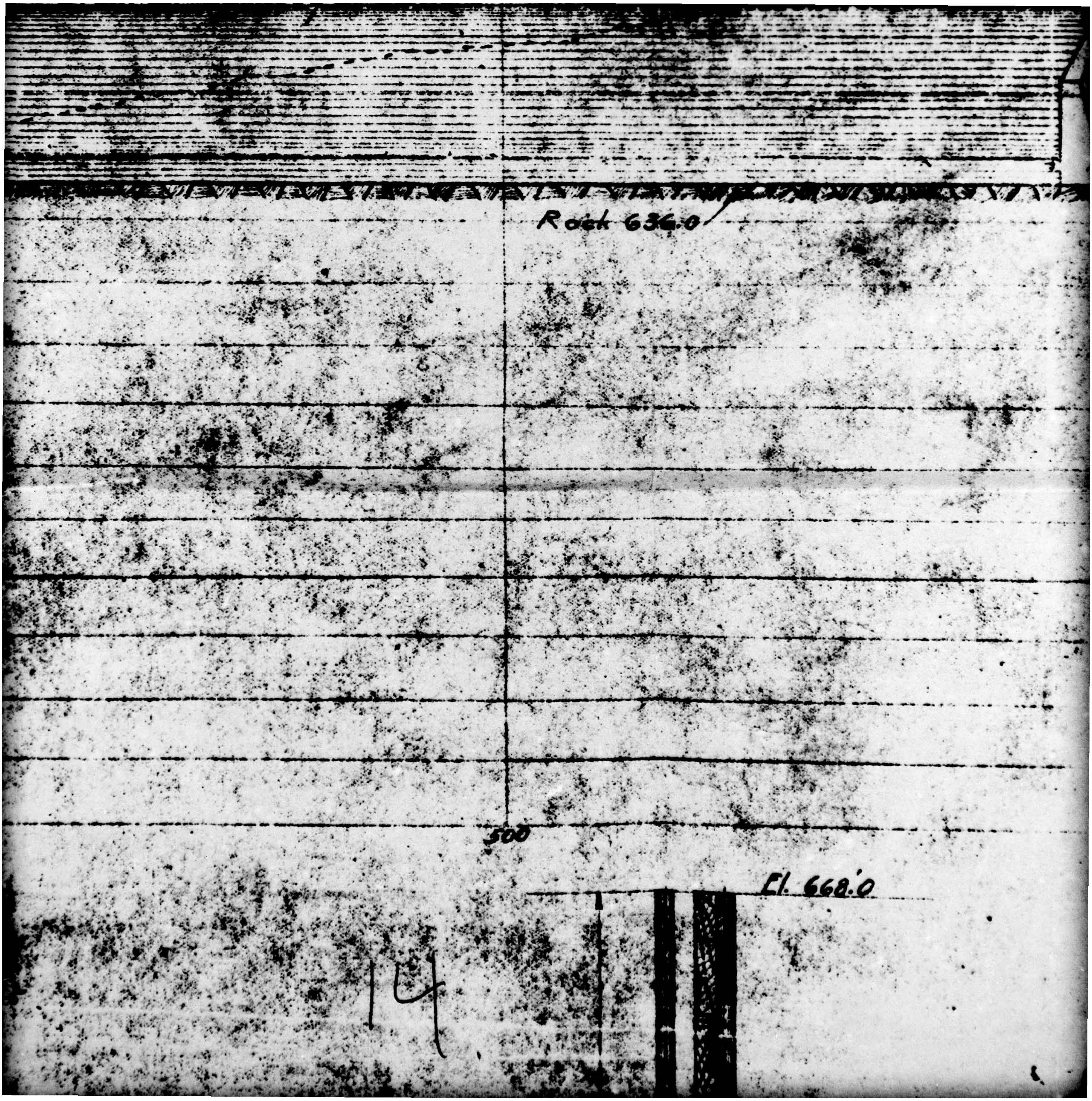
6

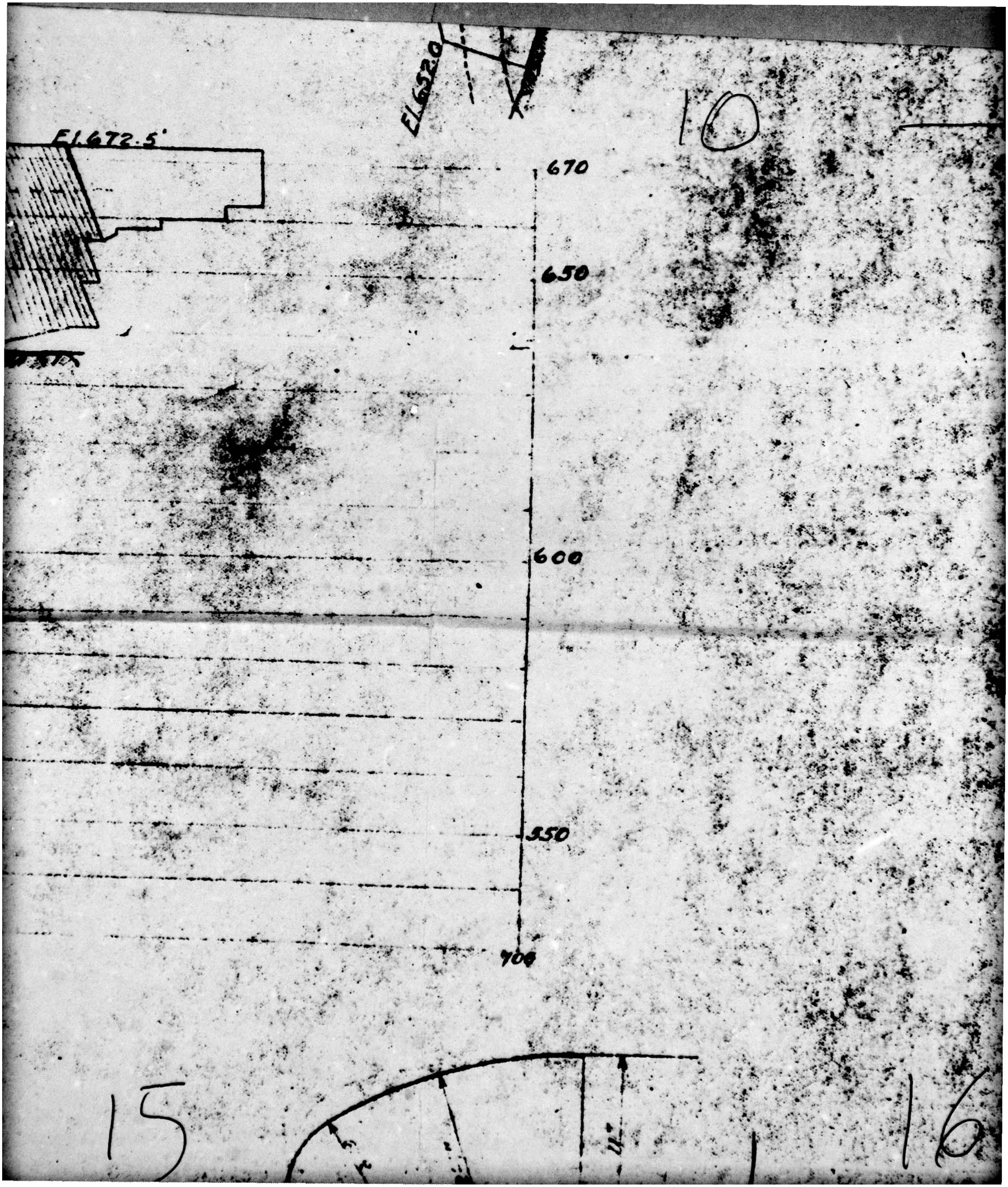

Corewall El. 672.5'
Rip Rap
To be located in field
-100
0
El. 672.5'
H.W. El. 669.0
For detail of Parapets notes etc. see dwg. C-3530
El. 660.0
12.5' wide
El. 647.5'

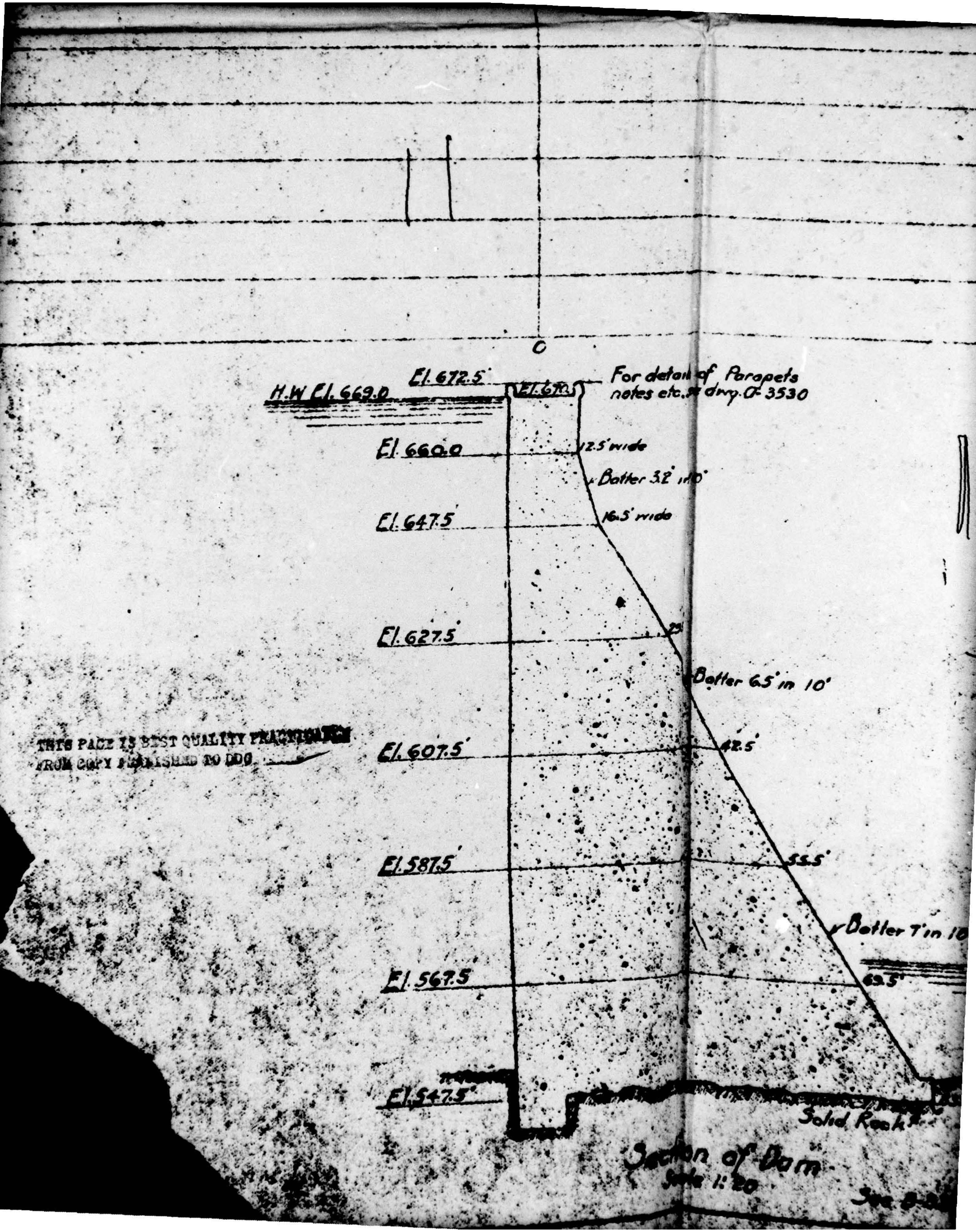


7
Elev. Top of Parapet
100
200
H.W. El. 669.0
12 Equal spaces
See Detail
El. 664.0
12

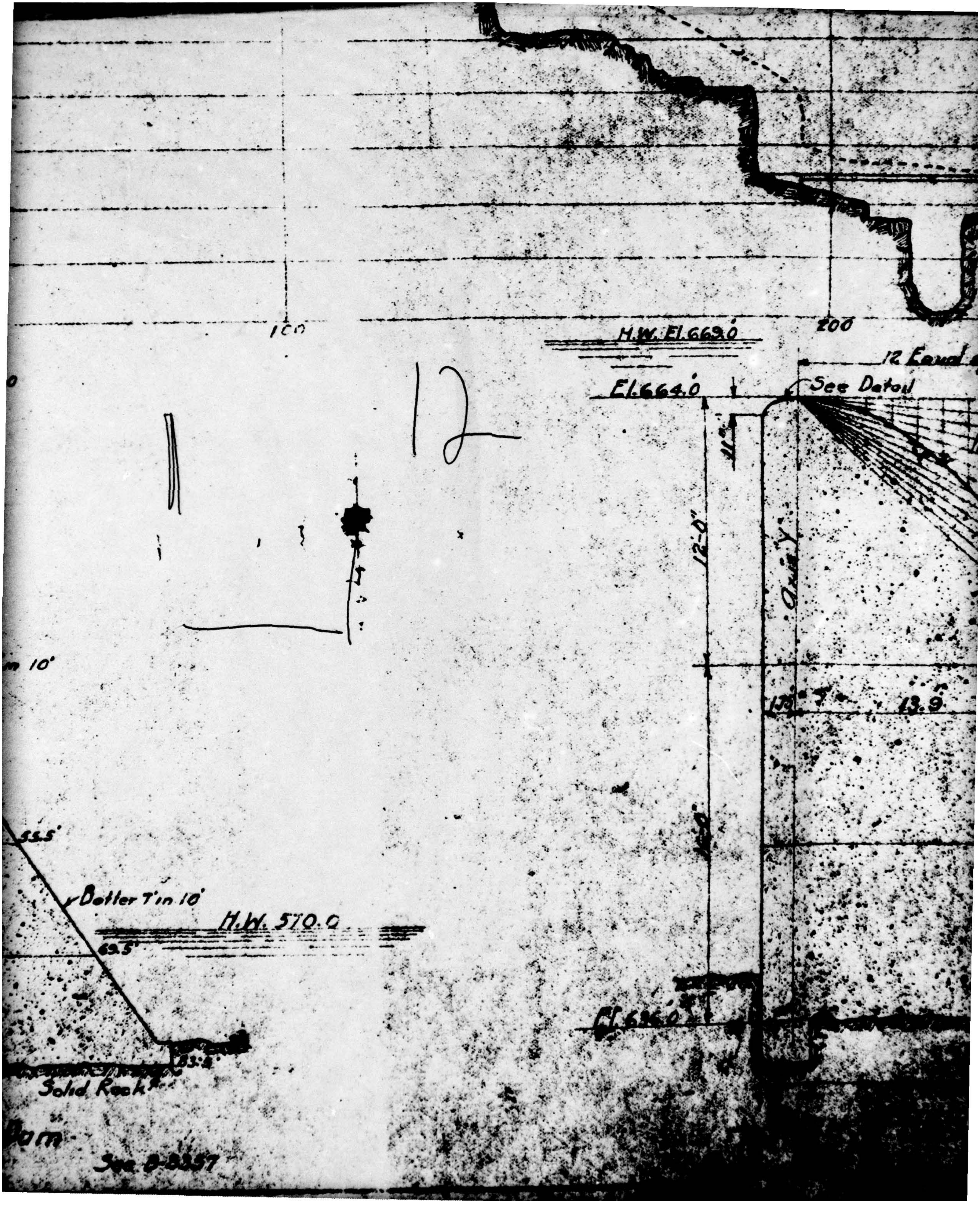

300
400
20


Rock 636.0
500
El. 668.0
14

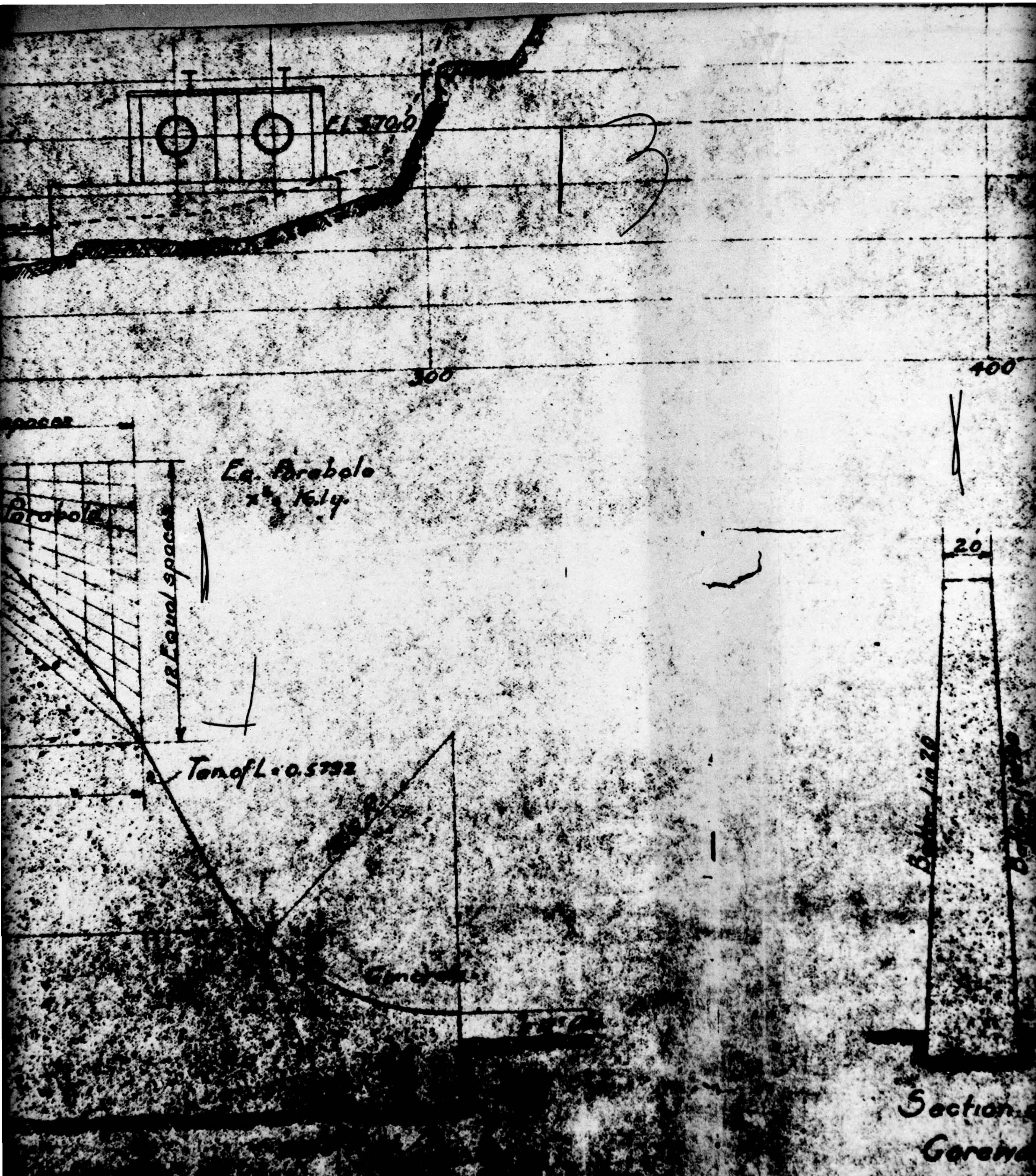

El. 672.5'
El. 652.0
10
670
650
600
550
700
15
16



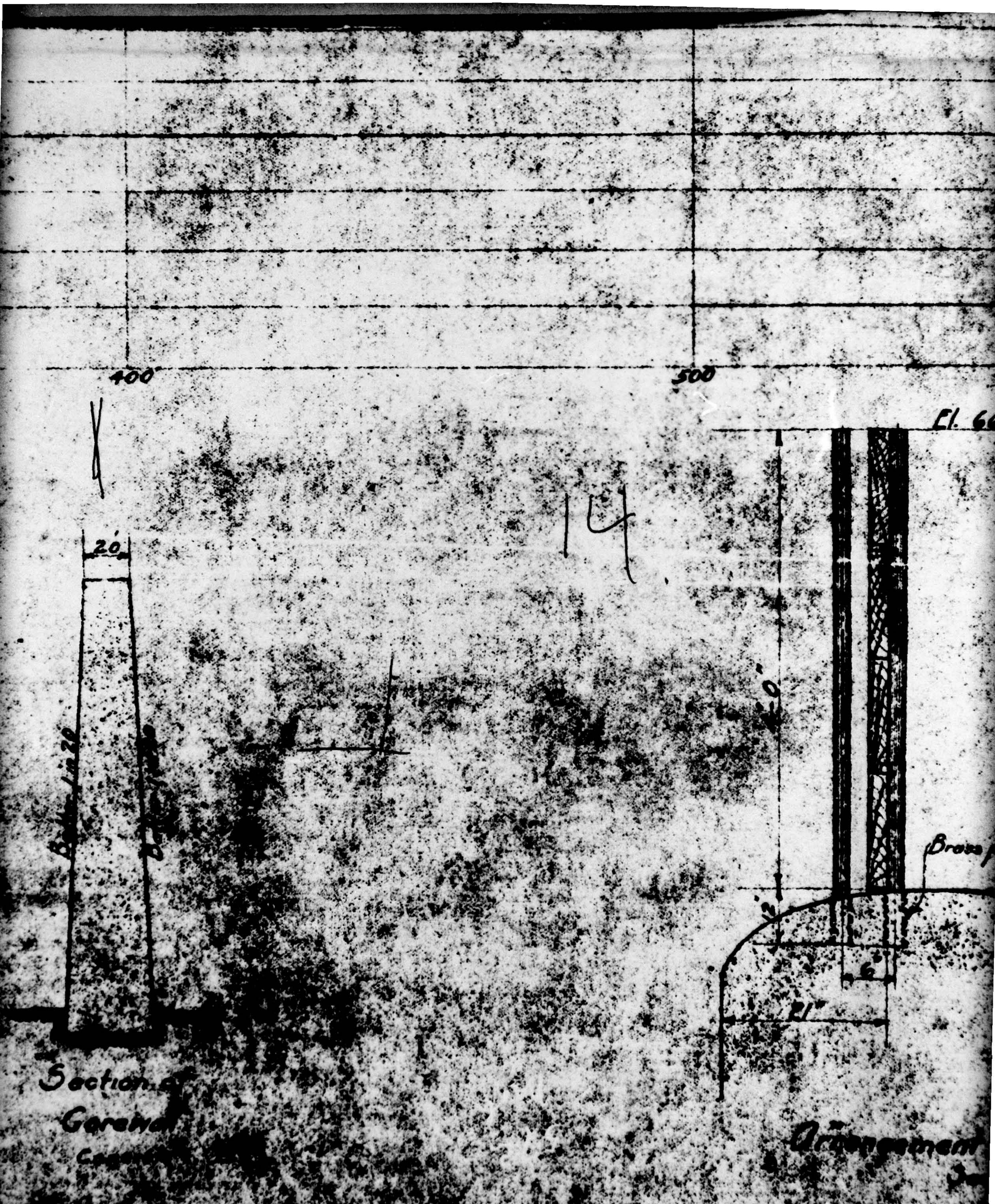
Section of Dam
Scale 1: 20

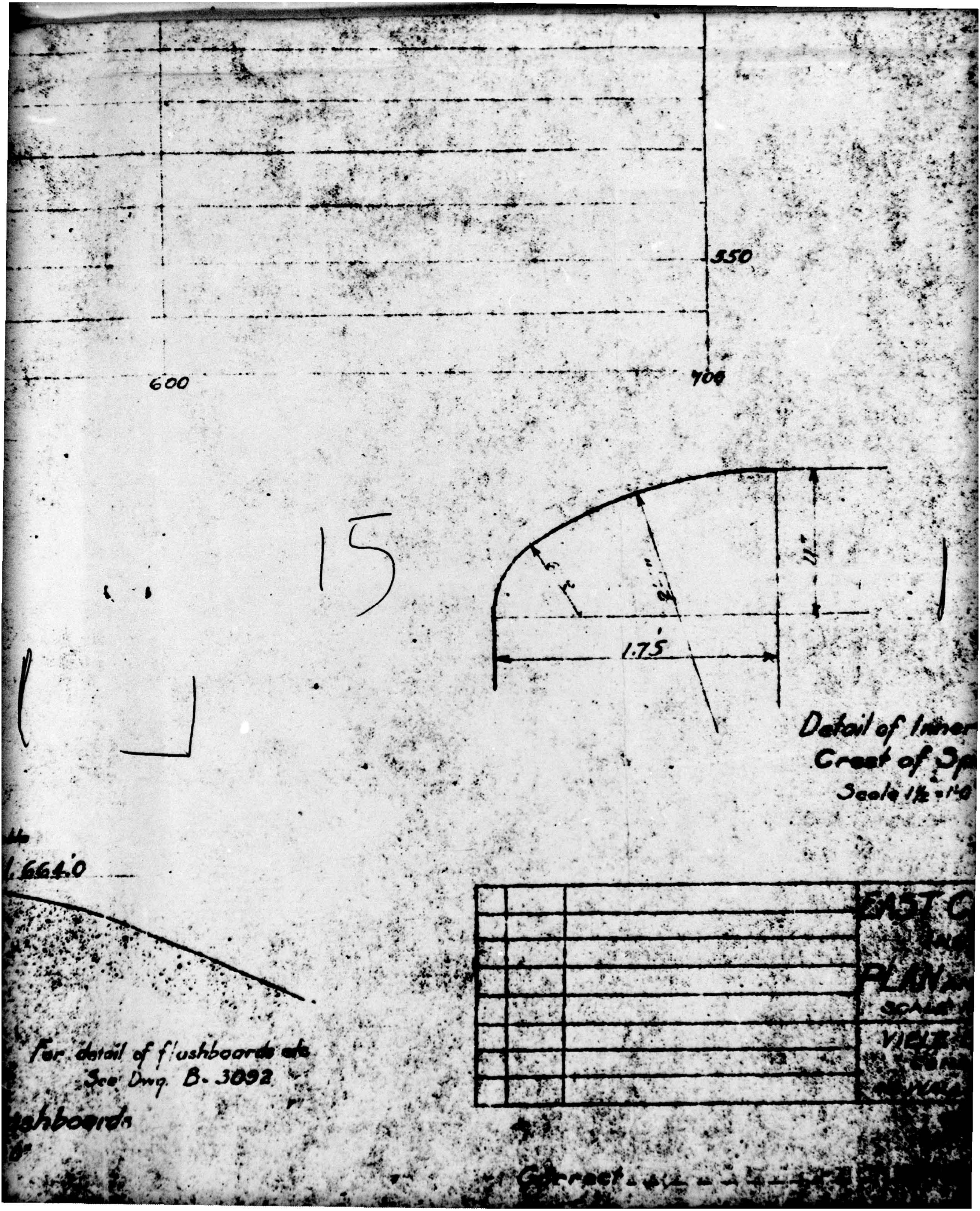


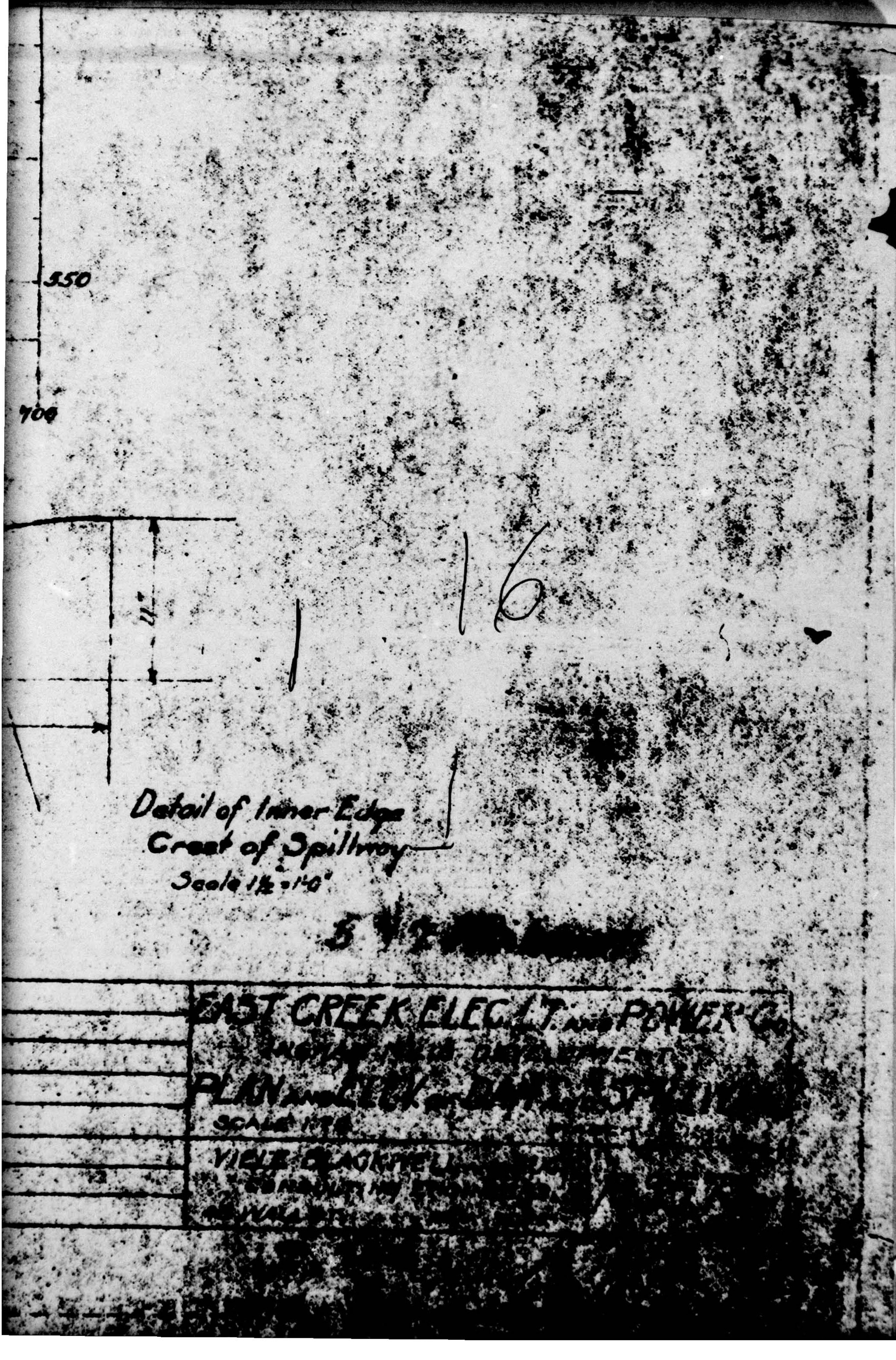
100
200
H.W. El.669.0
12 Equal
El.664.0
See Detail
12'-0"
Axis Y
13.9
in 10'
55.5'
Batter 7 in 10'
H.W. 570.0
63.5'
Solid Rock
Dam
See B-3357



El. 370.0
300
400
Eq. Parabola
18 Equal spaces
Parabola
Tan of L = 0.5732
20'
Section


400
500
El. 66
14
20
Brass
6
21
Section of
Corewall
Arrangement


350
600
700
15
1.75'
Detail of Inner
Crest of Sp
1,664.0
For detail of flushboards etc
See Dwg. B-3092
shboards
EAST C
PLAN
Correct


550
700
16
Detail of Inner Edge
Crest of Spillway
Scale 1½"=1'0"
EAST CREEK ELEC. LT. and POWER Co.
PLAN and